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TIMBER ENGINEERING
FOR DEVELOPING COUNTRIES

Part 2

Structural Timber and Products*

Prepared by

Agro-industries Branch,
Division of Industrial Operations

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PREFACE

The United Nations Industrial Development Organization (UNIDO) was established in 1967 to assist developing countries in their efforts towards industrialization. Wood is a virtually universal material which is familiar to people world-wide, whether grown in their country or not. Wood is used for a great variety of purposes but principally for construction, furniture, packaging and other specialized uses such as transmission poles, railway sleepers, matches and household woodenware. UNIDO has the responsibility within the United Nations' system for assisting in the development of secondary woodworking industries, and has done so since its inception, at national, regional and interregional levels through projects both large and small. UNIDO also assists through the preparation of a range of manuals dealing with specific topics of widespread interest which are common to most countries' woodworking sectors.[1/]

The lectures comprising this set of documents are part of UNIDO's continuing efforts to help engineers and specifiers appreciate the role that wood can play as a structural material. Part 2 consists of 8 out of the 36 lectures prepared for the Timber Engineering Workshop (TEW) held 2 - 20 May 1983 in Melbourne, Australia. The TEW was organized by UNIDO with the co-operation of the Commonwealth Scientific and Industrial Research Organization (CSIRO) and funded by a contribution made under the Australian Government's aid vote to the United Nations Industrial Development Fund. Administrative support was provided by the Australian Government's Department of Industry and Commerce. The remaining lectures are reproduced as Parts 1 and 3 to 5 covering a wide range of subjects, including case studies, as shown in the list of contents.

[1/] A fuller summary of these activities is available in a brochure entitled "UNIDO for Industrialization, Wood Processing and Wood Products", PI/78.

These lectures were complemented by site and factory visits, discussion sessions and assignment work done in small groups by the participants following the pattern used in other specialized technical training courses in this sector - notably in furniture and joinery production[1/] and on criteria for the selection of woodworking machinery[2/].

It is hoped that publication of these lectures will contribute to greater use of timber as a structural material to help satisfy the tremendous need for buildings: domestic, agricultural, industrial and commercial as well as for particular structures, such as bridges, in the developing countries. It is also hoped that this material will be of use to teachers in training institutes as well as to engineers and architects in both public and private practice.

Readers should note that examples cited are often of Australian conditions and may not be wholly applicable to developing countries despite the widespread use of the Australian timber stress grading and strength grouping systems and the range of conditions encountered in the Australian subcontinent. Readers should also note that the lectures were usually accompanied by slides and other visual aids, together with informal comments by the lecturer, for added depth of coverage.

The views expressed are those of the individual authors and do not necessarily reflect the views of UNIDO.

[1/] Lectures reproduced as ID/108/Rev.1.

[2/] Lectures reproduced as ID/247/Rev.1.

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INTRODUCTION

Many developing countries are fortunate in having good resources of timber but virtually all countries make considerable use of wood and wood products, whether home-grown or imported, for housing and other buildings in both structural and non-structural applications, as well as for furniture and cabinet work and specialized uses. It is a familiar material, but one that is all too often misunderstood or not fully appreciated since wood exists in a great variety of types and qualities.

There are certain well-known species that almost everyone knows of, such as teak, oak and pine, while some such as beech, eucalyptus, acacia, mahogany and rosewood are known primarily in certain regions. Others have been introduced to widespread use more recently, notably the merantis, lauans and keruing from Southeast Asia. Plantations also provide an increasing volume of wood. Very many more species exist and are known locally and usually used to good purpose by those in the business.

The use of timber for construction is not new and, in fact, has a very long tradition. This tradition has unfortunately given way in many countries to the use of other materials whose large industries have successfully supported the development of design information and teaching of engineering design methods for their materials - notably concrete, steel and brick. This has not been so much the case for timber despite considerable efforts by certain research and development institutions in countries where timber and timber-framed construction has maintained a strong position. Usually their building methods are based on the use of only a few well-known coniferous (softwood) species and a limited number of standard sizes and grades. Ample design aids exist and relatively few problems are encountered by the very many builders involved.

Recently, computer-aided design has been developed along with factory-made components and fully prefabricated houses with the accompanying improvement in quality control and decreased risk of site problems. Other modern timber engineering developments have enabled timber to be used with increasing confidence for an ever wider range of structures. This has been especially so in North America, Western Europe, Australia and New Zealand.

UNIDO feels that an important means of transferring this technology is through the organization of specialized training courses aimed at introducing engineers, architects and specifiers to the subject and especially drawing to their attention the advantages of wood (as well as disadvantages and potential problem areas) and reference sources so that for particular projects or structures, wood may be fairly considered in competition with other materials and used when appropriate. Cost comparisons, aesthetic and traditional considerations must naturally be made in the context of each country and project but it is hoped that the publication of these lectures will lead those involved to a rational approach to the use of wood in construction and remove some of the misunderstandings and misapprehensions all too often associated with this ancient yet modern material.

Material in this publication may be freely quoted or reprinted, but acknowledgement is requested together with two copies of the publication containing the quotation or reprint.

CHARACTERISTICS OF STRUCTURAL TIMBER

Robert H. Leicester[1]

1. INTRODUCTION

In design procedures, timber in treated in a manner similar to that of steel. However, there are at least three basic ways in which timber differs from steel with respect to its physical properties:

- the clear wood is essentially orthotropic
- structural timber contains natural defects which have very complex structural properties
- the properties of timber vary in a random manner from stick to stick and from one location to another within a stick.

(Other differences, such as the creep characteristics of timber, will be discussed in other sections.)

Because of the above, it will be found that there are significant differences between the structural characteristics of the following forms of timber:

- small clear pieces of wood
- clear structural size timber
- structural size timber containing natural defects
- pole timbers.

The differences between these forms of timber will be emphasised in the following discussion. Unless otherwise stated, all timber refers to sawn sticks of structural size.

2. ORTHOTROPIC ELASTICITY

The following applies to both small and large sizes of clear wood. The principal axes in timber lie along the longitudinal, radial and tangential directions as shown in Figure 1; these directions are denoted by L, R and T. Timber is considerably stiffer (and stronger) in the longitudinal direction than along any other principal axis.

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[1] An officer of CSIRO, Division of Building Research, Melbourne, Australia.

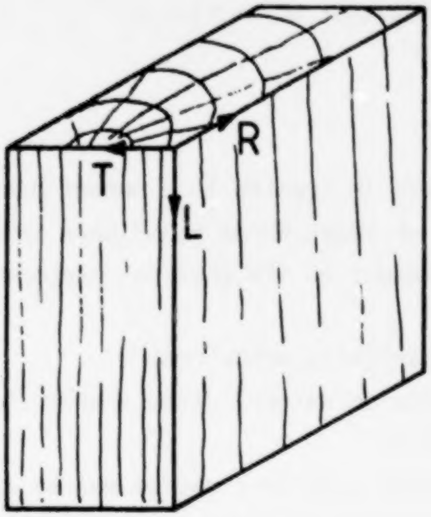


Figure 1 Principal axes in timber

Typical values of the elastic moduli along the principal axes, may be stated in terms of E_L, the stiffness along the longitudinal direction, are as follows:

$$E_R = 0.10\ E_L$$
$$E_T = 0.05\ E_L$$

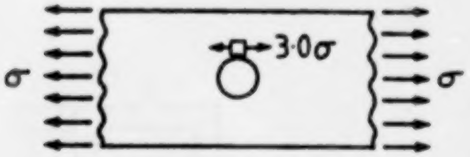
Similarly, typical shear moduli may be taken to be

$$G_{LT} = 0.060\ E_L$$
$$G_{LR} = 0.075\ E_L$$
$$G_{RT} = 0.018\ E_L$$

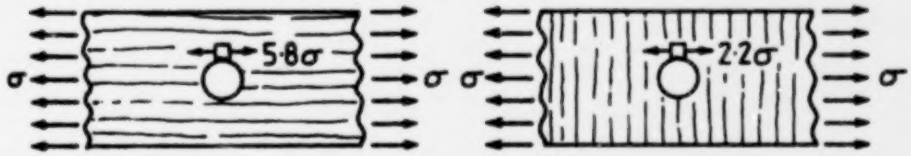
and typical Poisson's ratios are

$$\mu_{LR} = 0.40$$
$$\mu_{RL} = 0.04$$
$$\mu_{LT} = 0.40$$
$$\mu_{TL} = 0.10$$
$$\mu_{RT} = 0.50$$
$$\mu_{TR} = 0.25$$

It is outside the scope of this Section to analyse the various effects of orthotropicity; as an example, Figure 2 shows the effects of orthotropicity on stress concentrations at the edge of a circular hole.



(a) Isotropic material



(a) Wood stressed along the grain

(b) Wood stressed across the grain

Figure 2 Effect of orthotropicity on stress concentrations

3. SHEAR DEFORMATIONS

The following applies to both small and structural size timber. The low shear rigidity of timber leads to a greater proportion of deformation due to shear for wood structures in comparison with that of structures fabricated with isotropic materials such as steel. For example, the components of deflection at the centre of simply supported beams such as that shown in Figure 3 are as follows:

Deflection due to bending

$$\Delta_B = PL^3/(48\ EI) \qquad (1)$$

Deflection due to shear for solid members

$$\Delta_S = 1.5\ PL/(4\ G\ b\ d) \qquad (2)$$

Deflection due to shear for a box beam

$$\Delta_S = PL/(4\ G\ d_w\ \Sigma b_w) \qquad (3)$$

where the notation used is indicated in Figure 3.

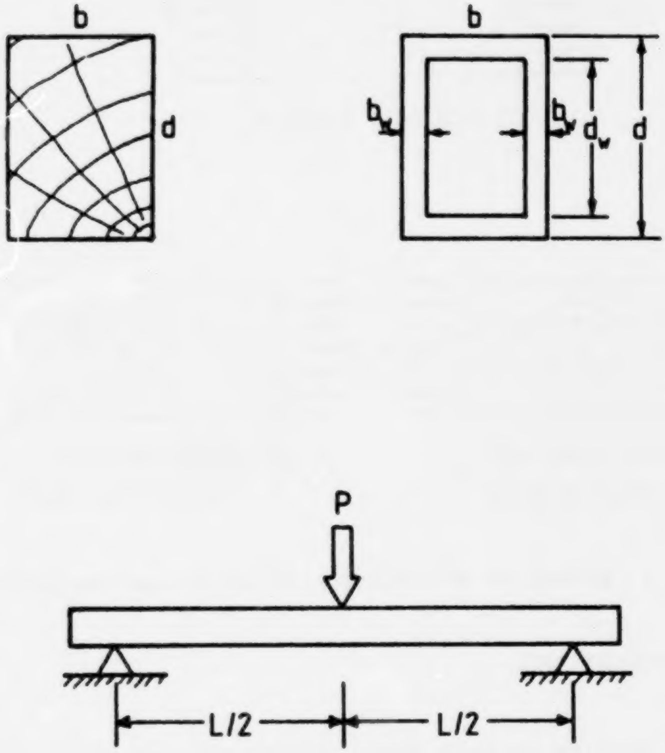


Figure 3 Notation used in deflection computation

For the typical examples of a solid timber and a box beam shown in Figure 4, the computed deflections due to shear are 8 per cent and 39 per cent respectively of the total deflection. In the computation, the shear modulus has been taken to be given by $G = 0.06\ E_L$.

Thus in contrast to the case for orthotropic materials, the shear deformation of beams are significant.

(Note. In most design codes, the design value of the specified modulus of elasticity for structural timber includes an allowance for shear deformations.)

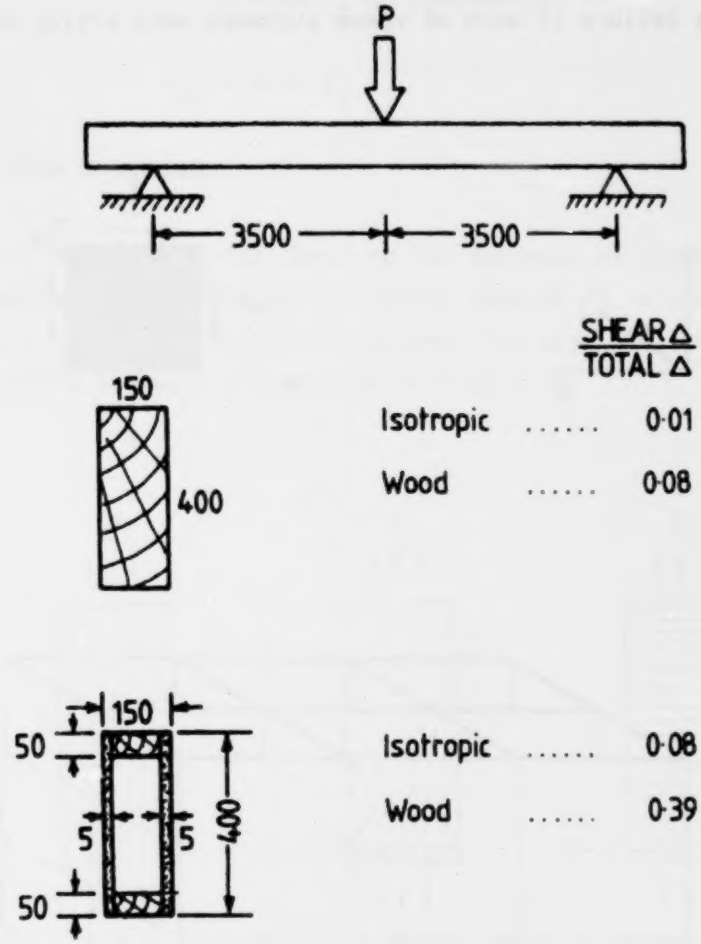

P
3500
3500
SHEAR Δ / TOTAL Δ
150
400
Isotropic ...... 0·01
Wood ...... 0·08
150
50
5
5
400
50
Isotropic ...... 0·08
Wood ...... 0·39

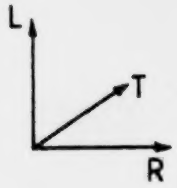
Figure 4 Examples of shear deflections

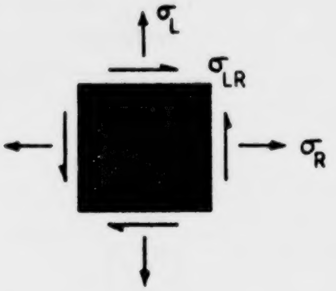
4. COMBINED STRESSES FOR CLEAR WOOD

Some ideas on a theory of wood strength can be derived by considering an idealised cellular structure aligned with the principal axes as shown in Figure 5. If it is assumed that the individual plates in this structures obey the von Mises failure criterion, then it can be shown that the failure criterion for the wood structure is given by three equations of the following type (Norris 1962),

$$(\sigma_L/F_L)^2 + (\sigma_R/F_R)^2 - (\sigma_L/F_L)(\sigma_R/F_R) + (\sigma_{LR}/F_{LR})^2 \leq 1 \qquad (4)$$

where σ_L, σ_R and σ_{LR} denote the applied stresses relative to the R and L axes, and F_L, F_R and F_{LR} are the corresponding values of these stresses that would cause failure if each of these stresses were acting on their own.





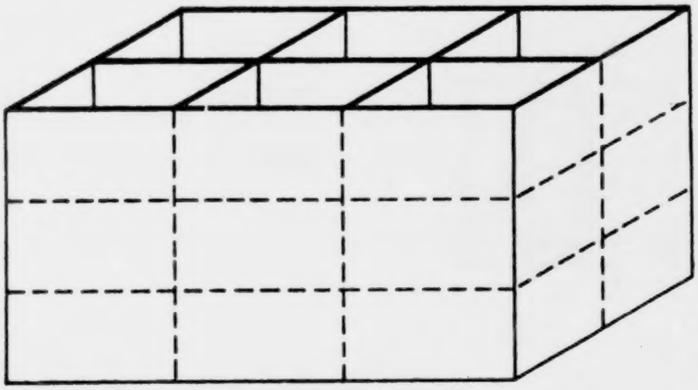


Figure 5 Idealised cellular structure

An approximation to equation (4) which has been found to fit the limited experimental data equally well is the following

$$(\sigma_L/F_L)^2 + (\sigma_R/F_R)^2 + (\sigma_{LR}/F_{LR})^2 \leq 1 \qquad (5)$$

Some typical relative values of the ultimate strength parameters are

F_L = 3.0 F_c in tension
= 1.0 F_c in compression

$F_R = 0.07\ F_c$ in tension
$ = 0.10\ F_c$ in compression

$F_{RL} = 0.20\ F_c$

5. HANKINSON'S FORMULA

Equation (4) may be used for deriving the strength of timber when the load is applied at an angle to the grain. However it is considerably simpler, and usually sufficiently accurate for practical purposes, to use Hankinson's formula, illustrated in Figure 6.

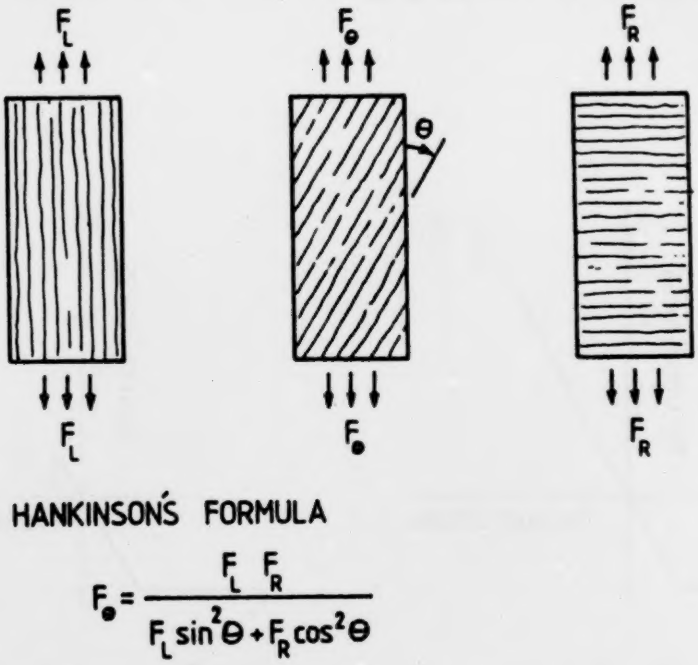

F_L
F_θ
F_R
θ
F_L
F_θ
F_R
HANKINSON'S FORMULA
$F_\theta = \frac{F_L F_R}{F_L \sin^2\theta + F_R \cos^2\theta}$

Figure 6 Illustration of Hankinson's formula

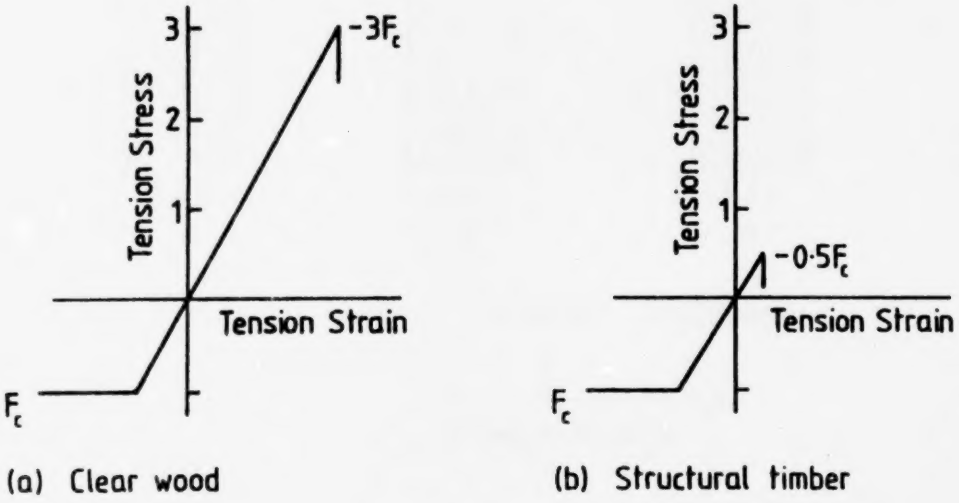
According to Hankinson's formula, the strength of wood at an angle θ to the direction of the grain will be denoted by F_θ and is given by

$$F_\theta = F_L\ F_R/(F_L\ \sin^2\theta + F_R\ \cos^2\theta) \tag{6}$$

Hankinson's formula has been found useful as a general method of interpolation to obtain estimates of a structural property at an angle θ to the wood grain. For example, it is usually applied to structural connectors.

6. SIMPLE MODELS OF BENDING STRENGTH

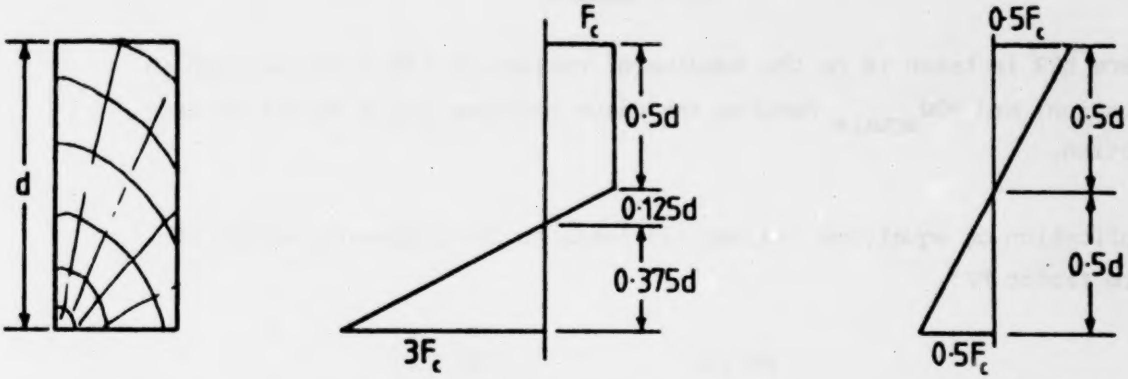
Bending strength involves a complex interaction of the tension and compression properties of wood. Some insight into the characteristics of bending strength may be obtained by considering the idealised stress-strain relationship shown in Figure 7. For both clear wood and structural timber the tension strength is taken to be brittle and the compression strength to be elasto-plastic. The tension strength is larger than the compression strength in the case of clear wood, and smaller in the case of structural timber. With these assumptions, the stress distribution at failure shown in Figure 8 is obtained.



(a) Clear wood

(b) Structural timber

Figure 7 Idealised characteristics of wood in direction of the grain



(a) Section

(b) Stress distribution in clear wood

(c) Stress distribution in structural timber

Figure 8 Stress-strain distribution at failure for rectangular members

Conventionally, the bending strength of timber beams is stated in terms of a modulus of rupture, denoted by 'MOR' and defined by

$$MOR = M_{ult}\, y_{max}/I \qquad (7)$$

where

M_{ult} = applied bending moment at failure

I = moment of inertia of the cross-section

y_{max} = maximum distance from the neutral axis to the edge

With this definition, the following MOR values are derived:

Section	MOR for clear wood	MOR for structural timber
square	2.00 F_c	1.00 F_t
round	2.15 F_c	1.00 F_t
I-beam	1.42 F_c	1.00 F_t

where the webs and flanges of the I-beam are taken to be 0.1 the beam width and depths respectively.

A form factor, denoted by FF, will be defined as follows

$$FF = MOR/MOR_{square} \tag{8}$$

where MOR is taken to be the modulus of rupture of the cross-section in question, and MOR_{square} denotes the value obtained for a square cross-section.

Application of equations (6) and (7) leads to the following values of form factor FF:

Section	FF for clear wood	FF for structural timber
square	1.00	1.00
round	1.07	1.00
I-beam	0.71	1.00

7. COMBINED BENDING AND AXIAL LOAD

A reasonable picture of the strength of timber members subjected to combined bending and axial load can be obtained through use of the idealised stress-strain curves shown in Figure 7. The resulting interaction curves are shown in Figure 9. The true relationship is difficult to measure experimentally. Useful data for the case of tension and bending loads on structural timber has been given by Senft and Suddarth (1970) and Senft (1973). One practical aspect noted in these studies was that the axial tension force has a significant effect on modifying the bending moment. A rough estimate of the effective bending moment M_{eff} is given by

$$M_{eff} = M_{nom} - (2/3)\ T\ \Delta_{nom} \tag{9}$$

where T is the applied axial tension force, M_{nom} is the nominal value of the applied bending moment, Δ_{nom} is the value of the deflection at the centre of the beam, computed assuming $T = 0$.

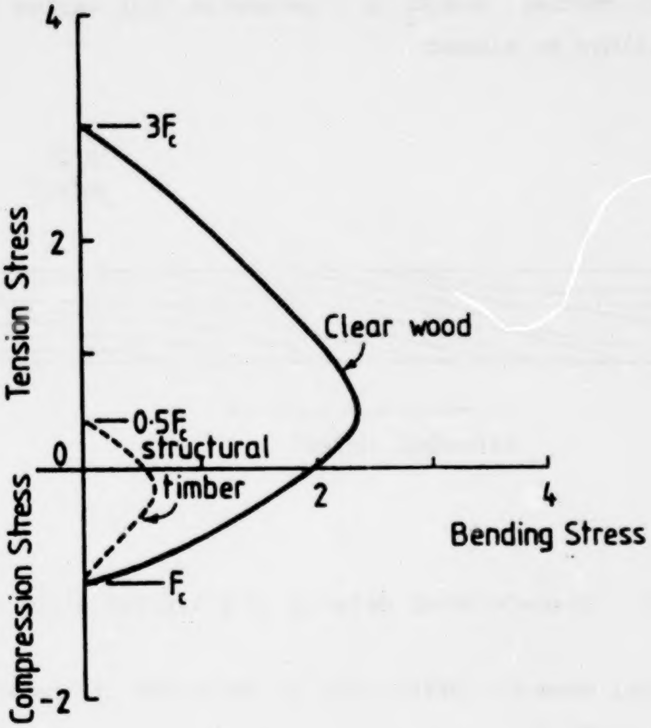


Figure 9 Interaction curves for combined bending and axial load

8. EFFECT OF NON-HOMOGENEITY

Unlike many conventional structural materials such as steel, the properties of timber can vary considerably within a single element. For example, a stick of lumber contains both strong clear wood and weak defects as indicated in Figure 10. This type of marked non-homogeneity has a significant effect on the characteristics of nominal strength.

A simple structural model of the nominal strength R of a particular structural element can be written

$$R = R_1 R_2 \tag{10}$$

$$\bar{R}_2 = 1.0 \tag{11}$$

where R_1 is a constant for any given structural element, but varies from one element to another, and R_2 is a parameter that varies from location to location within an element.

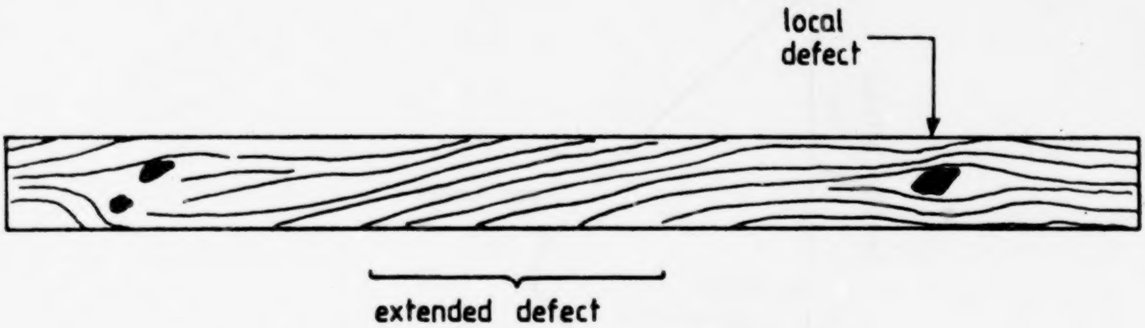


Figure 10 Dispersion of defects in a typical stick of timber

The conventional measure coefficient of variation of the structural element V_R is given by

$$V_R^2 = V_1^2 + V_2^2 \qquad (12)$$

where V_1 and V_2 are the coefficients of variation of R_1 and R_2 respectively. The simplest method of measuring V_2 is to cut pairs of test samples from each structural element, and then to measure the correlation coefficient r of the strength for these pairs of test samples. The coefficient of variation V_2 is then given by

$$V_2^2 = V_R^2 (1 - r) \qquad (13)$$

Details on the method of assessing the effects of non-homogeneity are given in Appendix B. Some simple practical examples will be given here.

For the case of geometrically similar elements that are 'brittle' so that failure eventuates if any internal flaw fails, the strength is given by

$$\bar{R} = \lambda_o \phi^{-V_2} \qquad (14)$$

where λ_o is a material and configuration constant, and ϕ denotes the volume of stressed material.

Equation (14) is a typical 'weakest link' relationship of the type first studied by Weibull (1939). Similar relationships can be derived for the case where the failure is related to the weakest area, or to the weakest cross-section of a beam. Figure 11 shows a size effect measured for the shear strength of timber elements. Other useful examples have been given by Foschi and Barrett (1976) and Bohannon (1966).

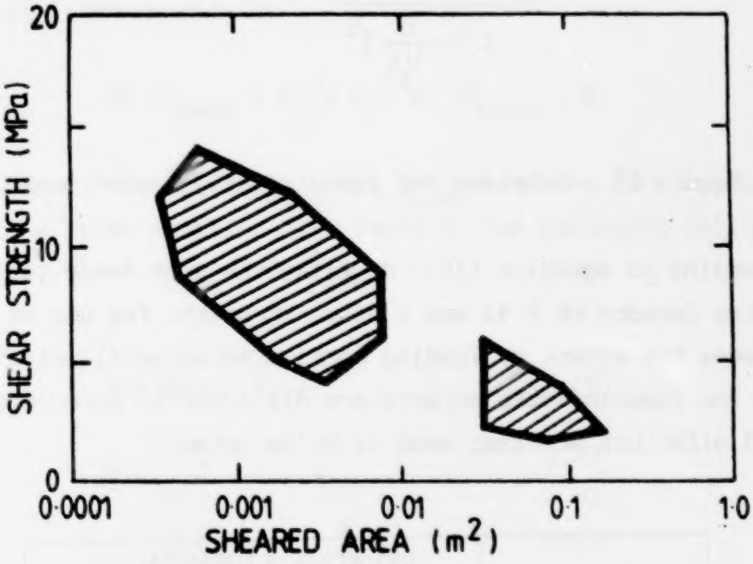


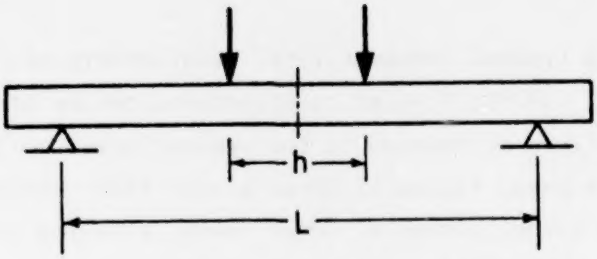
Figure 11 Effect of size on the shear strength of timber beams and glued joints (after Keenan 1974)

Another example of practical value is the symmetrically loaded beam shown in Figure 12. The strength of this beam is given by

$$\bar{R} = \lambda_{oo} (L [1 + (h/V_2 L)])^{-V_2} \qquad (15)$$

where λ_{oo} is a material constant.

For clear wood and structural timber, typical values of V_2 are 0.1 and 0.25 respectively.



$$\bar{R} = \frac{A_{00}}{L[1+\frac{h}{V_2 L}]^{V_2}}$$

Figure 12 Notation for symmetrically loaded beam

Hence, according to equation (15), doubling the span leads to the corresponding factors of 0.93 and 0.84 on strength. The use of equation (15) to assess the effect of loading configuration is illustrated in Figure 13. The fact that the effects are different in structural timber as compared with that of clear wood is to be noted.

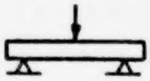
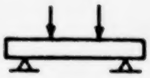
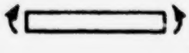
LOADING CONFIGURATION	RELATIVE STRENGTH	
	Clear Wood (V_2=0·10)	Structural Timber (V_2=0·25)
	1·00	1·00
	0·86	0·81
	0·79	0·67

Figure 13 Effect of load configuration on strength

The above theory has been stated in terms of mean nominal strength, but it also gives the same answers when applied to characteristic values, such as 5-percentile strength values.

Finally, mention should be made of a configuration factor that is peculiar to beams. Beam strength will vary depending on whether the edge placed in tension is randomly chosen or is deliberately selected to be the weakest edge. If the two values of nominal strength are denoted by R_{rand} and R_{weak} respectively, then for the weaker pieces of timber where defects are visually discernable, the relationship between these strengths is given roughly by

$$Pr\ (R_{weak} < x) \approx 1.5\ Pr\ (R_{rand} < x) \qquad (16)$$

If it is assumed that both R_{weak} and R_{rand} have the same types of distributions, then equation (16) leads to the following ratios of the 5-percentile values R^o_{weak} and R^o_{rand}:

Coeff. of variation V_R	R^o_{weak}/R^o_{rand}
0.1	0.97
0.2	0.93
0.3	0.89

In the context of the above discussion it is of interest to note the following methods that have been used in various countries to measure bending strength:

USA/Canada	... random location of defects, random edge placed in tension
UK	... weakest defect at maximum stress section, random edge placed in tension
Australia	... weakest defect at maximum stress section, weakest edge placed in tension.

9. EFFECT OF NATURAL DEFECTS

Natural defects introduce zones of weakness and sometimes zones of flexibility into structural timber. Some of the effects of knots and sloping grain are illustrated in Figures 14 and 15. Other defects that have been studied include splits, checks and kino veins, local pockets of decay, compression shakes, and bark inclusions.

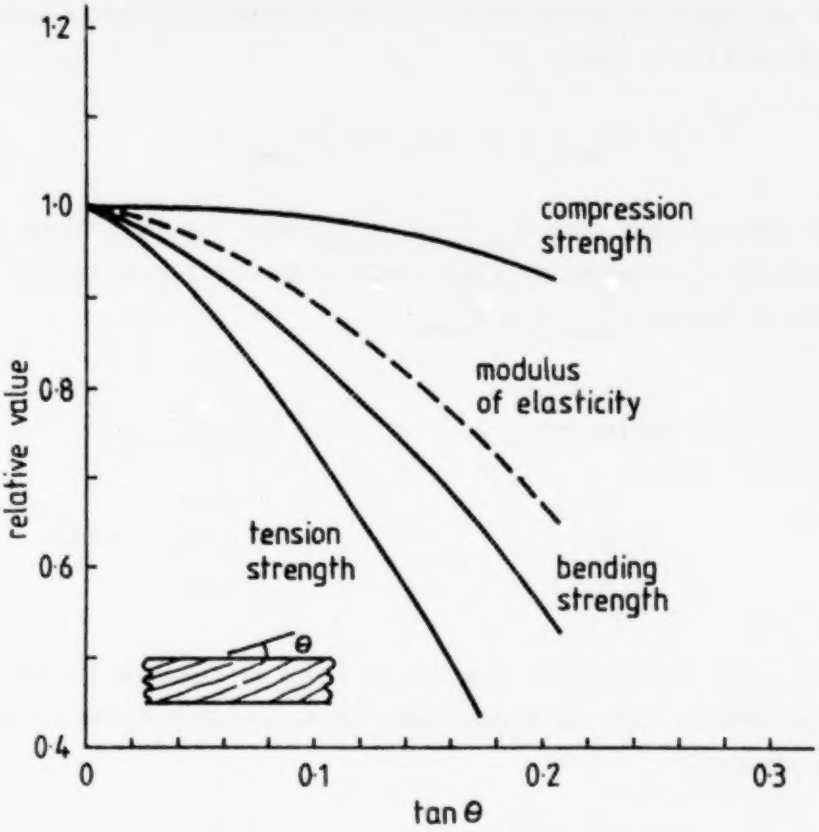


Figure 14 Effect of slope of grain on structural characteristics of timber

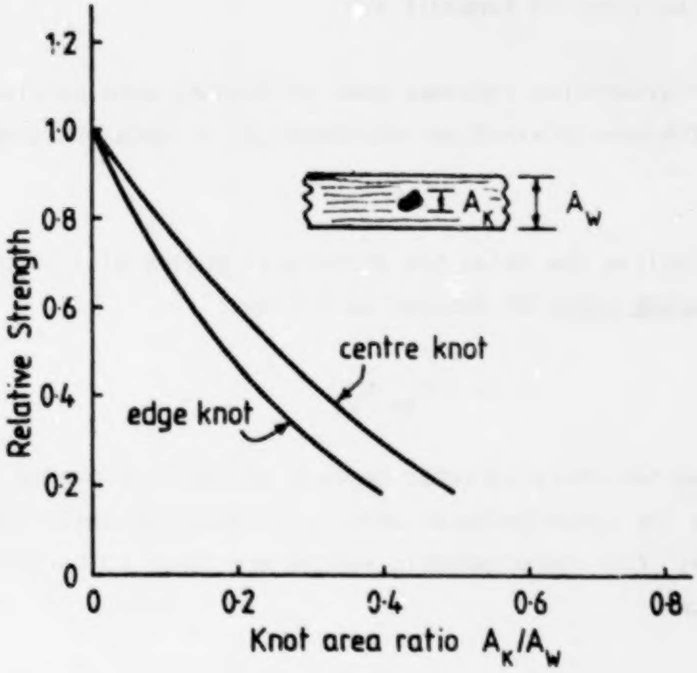


Figure 15 Effect of knot size on tension strength (after Dawe 1964)

10. CHARACTERISTICS OF GRADED TIMBER

Structural timber is sorted into grades by limiting the sizes of visual defects or some other grading parameters such as local stiffness. The structural characteristics of graded timmber can differ quite markedly from that of clear wood and therefore must be studied as a separate material. Three excellent sources of published data on the properties of graded timber are the following:

(a) Southern pines of USA ... Doyle and Markwardt (1966, 1967)

(b) Timber of western Canada ... McGowan et al. (1977), Littleford (1978), Littleford and Abbott (1978)

(c) Imported UK timber ... Curry and Tory (1976), Curry and Fewell (1977), Fewell (1980).

A summary, based on the data by Doyle and Markwardt (1966, 1967) on southern pines is given in Appendix A.

The information given also includes some additional data on clear wood properties, which was obtained by extrapolation of data obtained from other sources.

In order to normalise the data, the structural grades will be quantified in terms of a grade ratio GR defined as follows:

$$GR = R_{kg}/R_{kc} \tag{17}$$

whre R_{kg} denotes the characteristic bending strength of graded material and R_{kc} denotes the characteristic bending strength of small clear wood specimens. Herein the characteristic values are taken to be the five percentile values.

For the data on Southern pines given in Appendix A, equation (17) leads to the following values of grade ratios:

Timber	Grade ratio
small clears	1.00
structural clears	0.72
No. 1 grade	0.63
No. 2 grade	0.39
No. 3 grade	0.28

Figures 16 to 18 shows the effect of grade ratio on some of the properties of Southern pine. It is important to note that the properties of small clear pieces of wood do not provide an accurate picture of the characteristics of structural size timber.

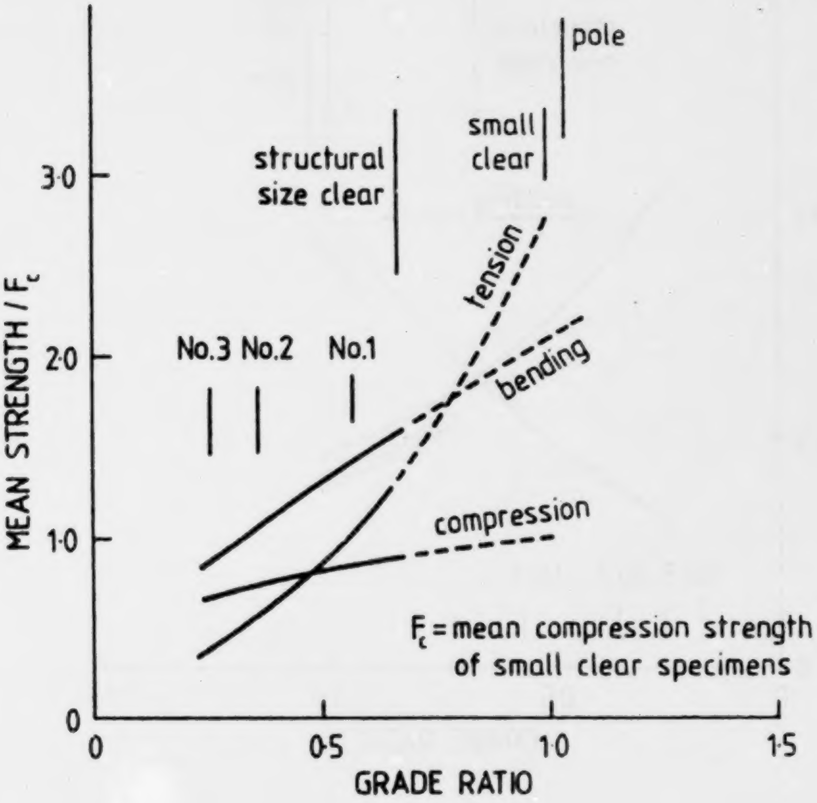


Figure 16 Effect of grade ratio on mean strength of Southern pine (after Doyle and Markwardt 1966, 1967)

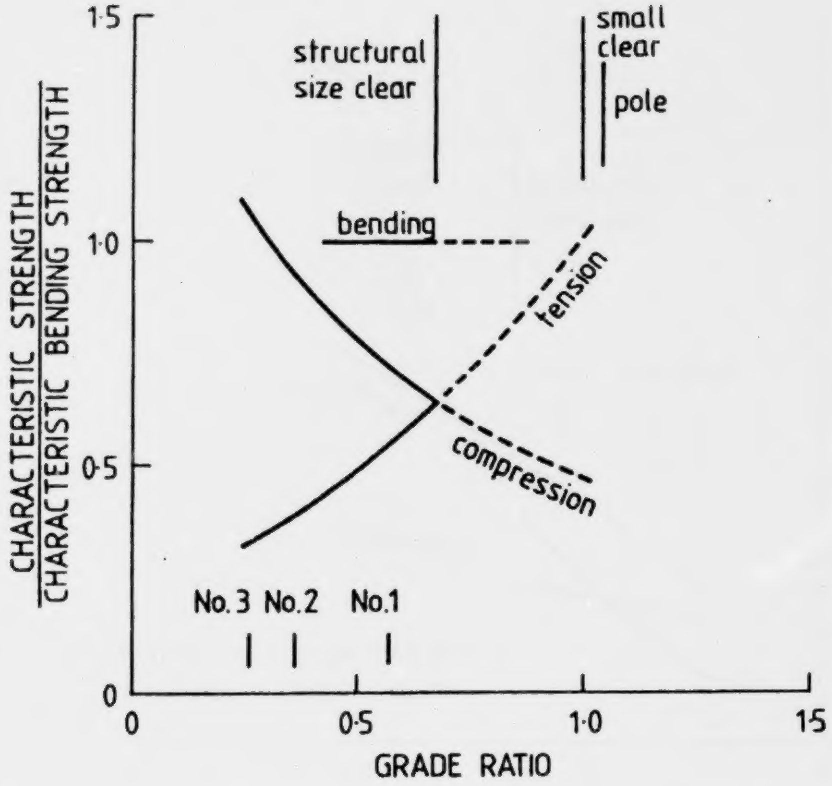


Figure 17 Effect of grade ratio on the characteristic strength ratio of Southern pine (after Doyle and Markwardt 1966, 1967)

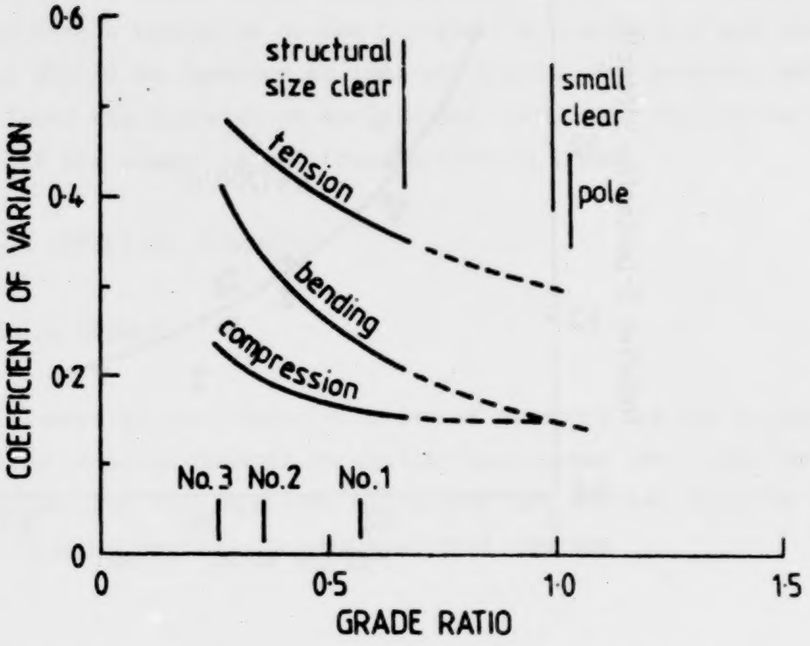


Figure 18 Effect of grade ratio on the coefficient of variation of Southern pine (after Doyle and Markwardt 1966, 1967)

One troublesome aspect of visually graded timber is that it exhibits a size effect. For sticks of timber with the same visually estimated bending strength ratio but of different size, there is a loss of strength as the stick becomes larger. As an example, sticks of the same grade of softwood timber, the characteristic strength values R_k have the following form:

Compression strength ... $R_k = \lambda_1$
Bending strength ... $R_k = \lambda_2/d^{0.4}$
Tension strength ... $R_k = \lambda_3/d^{0.2}$

wheee λ_1, λ_2 and λ_3 are constants, and d is the width of the member. The size effect for tension strength is illustrated in Figure 19. This size effect is probably related to the size effect of fracture that will be discussed elsewhere.

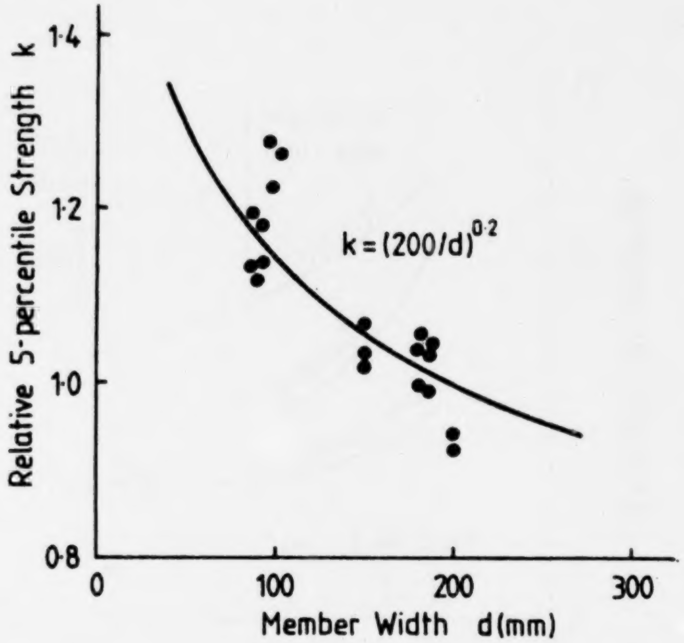


Figure 19 Effect of size on tension strength (after Curry and Fewell 1981).

Finally, mention should be made of the correlation between structural properties. This correlation is of importance in grading procedures. Higher correlation coefficients lead to more efficient grading. The following are some typical values of correlation coefficients between various parameters and bending strength.

Predictor for bending strength	Correlation coefficient r
density	0.50
average MOE	0.50
visual defect size	0.65
local MOE	0.85
visual defect size plus density	0.80
local MOE plus average MOE	0.90

These correlation coefficients are poorer than the values that are obtained with clear wood. A correlation coefficient of less than 0.6 is of little practical value for grading purposes. It should be noted that many factors affect the value of the correlation coefficient and the above values should be regarded as indicative only. For example, Anton (1979) has found the correlation coefficient with local MOE can be as low as 0.35 if the timber is cut from very young trees.

11. FACTORS AFFECTING STRENGTH

11.1 Seasoning Effects

Figure 20 illustrates the characteristics of strength during drying. The strength value obtained depends on whether the timber can 'case-harden' without degrade. For many species, sizes that are thicker than 150 mm in cross-section are difficult to season without degrade.

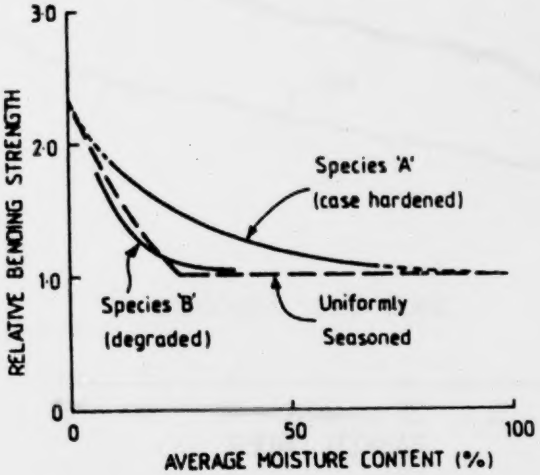


Figure 20 Strength furing drying (after Wilson et al. 1930)

11.2 Effect of Moisture Content

The effect of moisture content on strength is illustrated by the example given in Figure 21. The important matter to note is that while there is an increase in strength on drying for small clear pieces of wood, there is little improvement in strength for the weaker pieces of structural timber which determine the strength values that are recommended for design purposes.

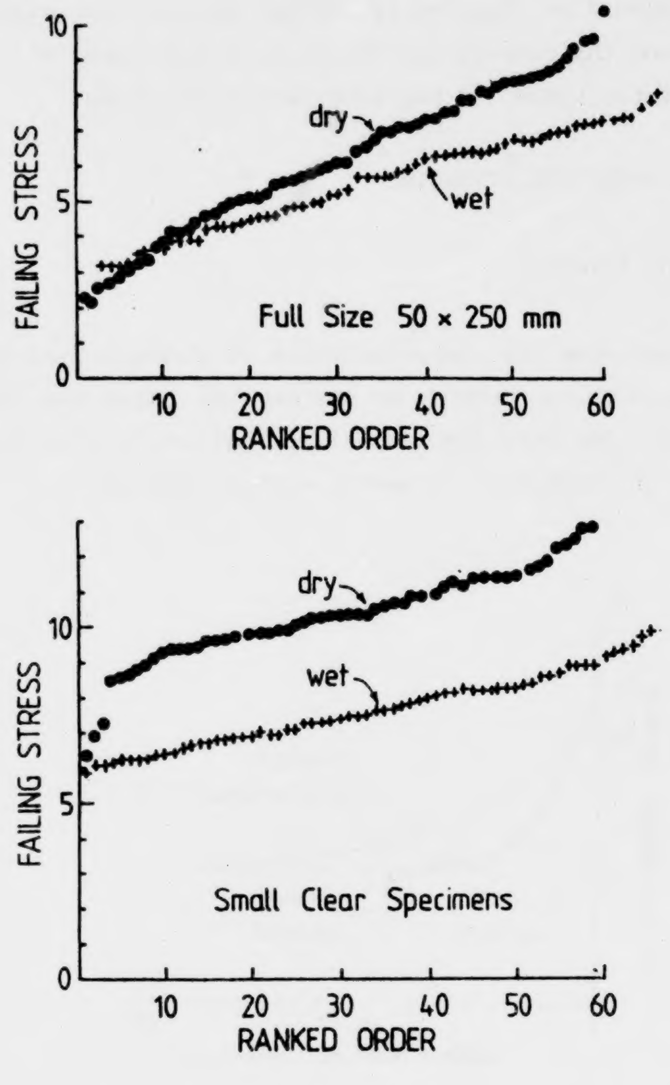


Figure 21 Effect of moisture content on the bending strength of Douglas-fir (after Madsen 1972)

11.3 Effect of Duration of Load

The effect of duration of load is illustrated by the example shown in Figure 22. Here again it is to be noted that while the average piece of structural timber exhibits a marked decrease in strength with load duration, only a limited effect is observed for the weaker sticks of timber which determine the strength values that are recommended for design purposes.

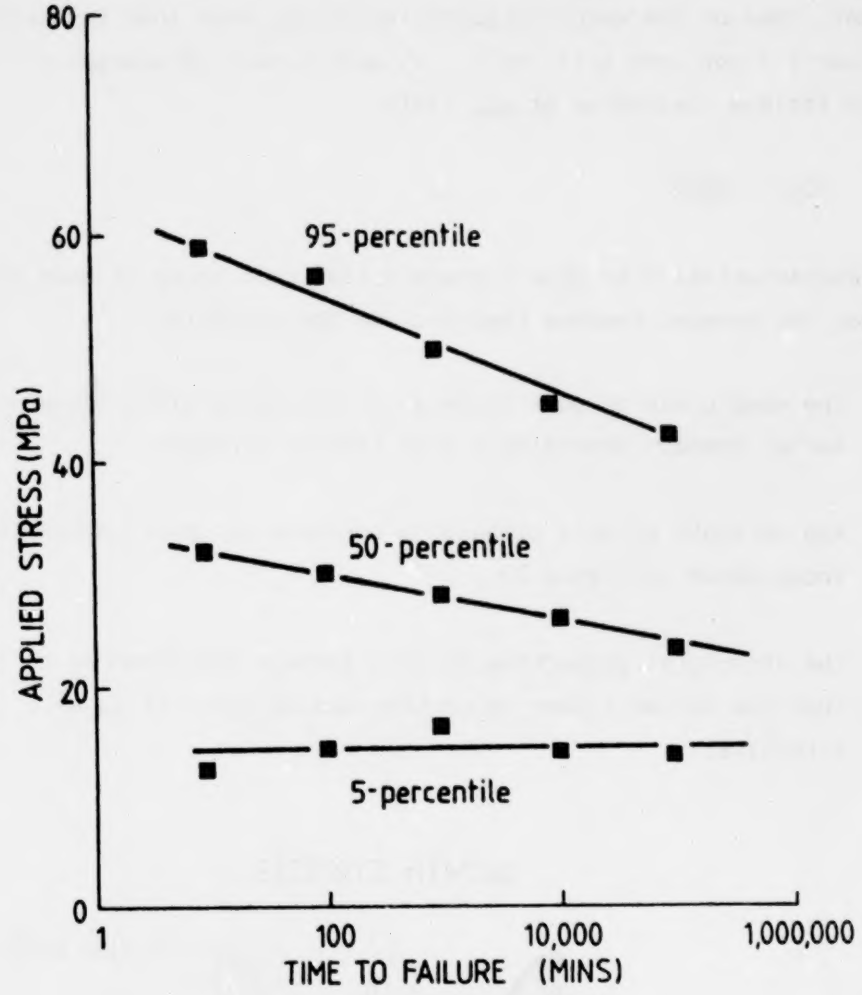


Figure 22 Bending strength of dry Hem-fir subjected to ramp loading (after Madsen 1971)

11.4 Fatigue Strength

The excellent fatigue strength of clear wood is well known, but it is not generally appreciated that structural timber containing natural defects can deteriorate due to fatigue.

In one set of tests on Radiata pine, it was found that the average strength was reduced by 20 per cent when subjected to 500 cycles of repeated loading. In other tests, it has been found that if a load is applied such that it fails 5 per cent of a population of structural timber, then on the second application of the same load typically a further 0.5 per cent will fail; i.e. some timber is susceptible to one cycle fatigue (Leicester et al. 1981).

12. POLE TIMBER

The characteristics of pole timbers differ from those of sawn structural timber for several reasons that include the following:

- the wood grain of pole timbers is continuous along edges and around knots, thereby providing a high tension strength
- the strength of pole timbers is enhanced by growth stresses such as those shown in Figure 23
- the structural properties of pole timbers are enhanced by the fact that the better timber is on the outside where it is more effective.

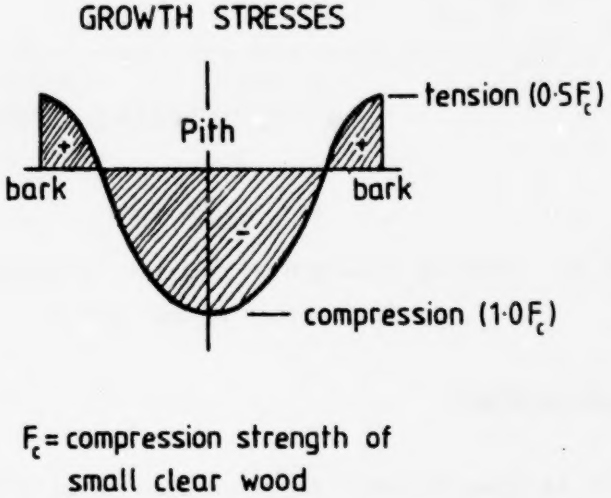


Figure 23 Growth stresses in a typical hardwood

In an extensive series of tests on poles, Boyd (1961 to 1968) found the following typical properties of pole timbers:

$$\text{mean}\ (F_b) = 1.1\ \text{mean}\ (F_{bsc}) \tag{18a}$$

$$\text{mean}\ (E) = 1.2\ \text{mean}\ (E_{sc}) \tag{18b}$$

$$\text{cov}\ (F_b) = 0.13 \tag{18c}$$

where F_b and E denote the bending strength and modulus of elasticity of the pole timbers in green moisture content conditions, and F_{bsc} and E_{sc} are the corresponding values measured on green, small clear specimens; the term cov () denotes the coefficient of variation. From the above the following is obtained

$$\text{grade ratio} = 1.1 \tag{19}$$

A comparison of this grade factor with those given earlier indicates that the effective grade of pole timbers is more than 50 per cent better than that which can be obtained for sawn timber.

Other matters of practical importance that were noted by Boyd include the following:

- the age of the tree can have a significant influence on the structural properties as illustrated by the example shown in Figure 24
- a regime of seasoning and resoaking, as would occur in the preservative treatment of poles, may reduce the strength by up to 15 per cent
- shaving poles to 'improve' their aesthetics could lead to a loss of some 10 per cent in strength; this is due to the interruption in the continuity of the wood grain.

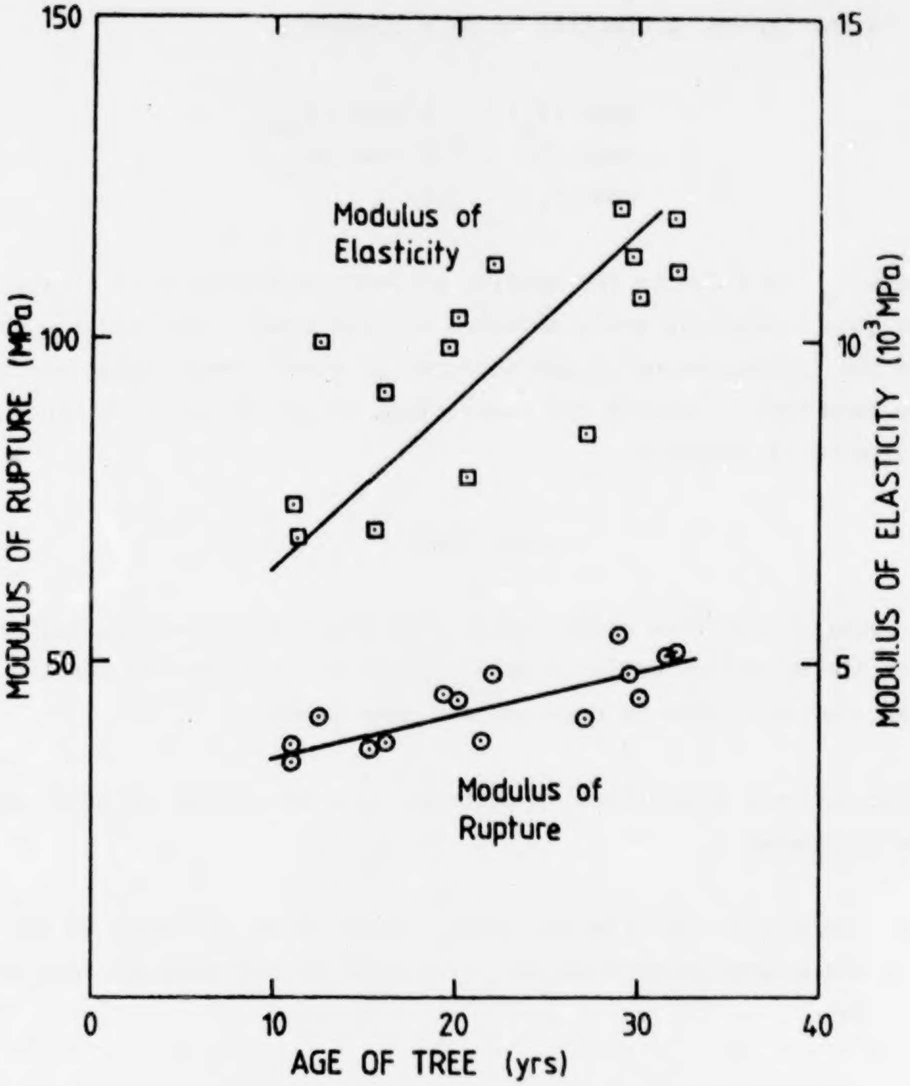


Figure 24 Effect of tree age on the pole properties of radiata pine (after Boyd 1964)

13. TAPERED BEAMS

For reasons of economy, beams are sometimes cut to a taper. The stresses that arise in this situation have been discussed by Macki and Kuenzi (1965). In addition to the normal checks on strength, an additional check should be made for strength to resist the combined stresses acting along the edge of the beam as shown in Figure 25. In this check the failure criterion given by equation (5) is to be used.

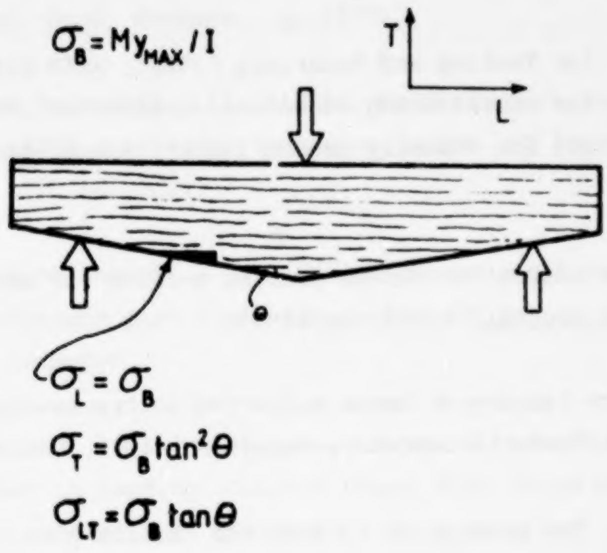

$\sigma_B = My_{MAX}/I$
T
L
θ
$\sigma_L = \sigma_B$
$\sigma_T = \sigma_B \tan^2\theta$
$\sigma_{LT} = \sigma_B \tan\theta$

Figure 25 Stresses along edge of tapered beam

14. CONCLUSIONS

The preceding discussion has provided information on the structural characteristics of timber that are relevant to the formulation of design recommendations. These include the following:

- both the orthotropic and the non-homogeneous nature of timber must be considered
- while the properties of small specimens of clear wood are useful as index properties for structural timber, it can be quite misleading to use the characteristics of these specimens to derive the characteristics of structural timber
- design strength is related to the five-percentile values of strength which may not have the same structural characteristics as the mean strength.

15. REFERENCES

American Society for Testing and Materials (1980). ASTM D245-74. Standard methods for establishing structural grades and related allowable properties for visually graded lumber. Annual Book. Part 22. USA.

Anon. (1972). New Australian stress grading machine for world markets. Australian Timber Journal, April, pp.65-69.

Anon. (1941). Form factors of beams subjected to transverse loading only. USDA Forest Products Labratory Report No. 1310, October.

Anton, A. (1979). The grading of 13-year old radiata pine. Proceedings of 19th Forest Products Research Conference, Melbourne, November.

Bohannon, B. (1966). Effect of size on bending strength of wood members. US Forest Service Research Paper FPL 56, May.

Boyd, J.D. (1961-1968). The strength of Australian pole timbers. Division of Forest Products Technological Papers. No.15 (1961, I. Messmate Stringybark poles), No. 22 (1962, II. Principles in derivation of design stresses for poles), No. 2 (1962, III. Jarrah poles), No. 32 (1964, IV. Radiata pine poles), No. 50 (1967, V. Yellow stringybark poles), No. 53 (1968, VI. Grey ironbark poles).

Curry, W.T., and Tory, J.R. (1976). The relation between the modulus of rupture (ultimate bending stress) and modulus of elasticity of timber. Current Paper 30/76. Building Research Establishment. Princes Risborough, UK, April.

Curry, W.T., and Fewell, A.R. (1977). The relations between the ultimate tension and ultimate compression strength of timber and its modulus of elasticity. Princes Risborough Laboratory, BRE, Bucks., UK, May.

Curry, W.T., and Fewell, A.R. (1981). Grade stress values for timber. Paper prepared for Economic Commission for Europe, Timber Committee, Geneva, June.

Dawe, P.S. (1964). The effect of knot size on the tensile strength of European redwood. Wood, November, pp.49-51.

Doyle, D.V., and Markwardt, L.J. (1966). Properties of southern pine in relation to strength grading of dimension lumber. US Forest Service Research Paper, FPL 64, July.

Doyle, D.V., and Markwardt, L.J. (1967). Tension parallel-to-grain properties of southern pine dimension lumber. US Forest Service Research Paper, FPL 84, December.

Fewell, A.R. (1980). Relations between the moduli of elasticity of structural timber in bending. Current Paper 9/80. Princes Risborough Laboratory, BRE, Bucks., UK, December.

Foschi, R.O., and Barrett, J.D. (1975). Longitudinal shear strength of Douglas-fir. Canadian Journal of Civil Engineering, Vol.3, No.2, pp.198-208.

Gerhards, C.C. (1970). Further report on seasoning factors for modulus of elasticity and modulus of rupture. Forest Products Journal, Vol.20, No.5, May, pp.40-41.

Gerhards, C.C., and Ethington, R.L. (1974). Evaluation of models for predicting tensile strength of 2- by 4-inch lumber. Forest Products Journal, Vol.24, No.12, December, pp.46-54.

Keenan, F.J. (1974). Shear strength of wood beams. Forest Products Journal, Vol.24, No.9, pp.63-70.

Leicester, R.H., Breitinger, H.O., and McNamara, R. (1981). Damage due to proof loading. Proc. of 20th Forest Products Research Conference, Topic E1, Melbourne, Australia, Nov.

Littleford, T.W. (1978). Flexural properties of dimension lumber for western Canada. Information Report VP-X-179. Western Forest Products Laboratory, Vancouver, Canada, July.

Littleford, T.W., and Abbott, R.A. (1978). Parallel-to-grain compressive properties of dimension lumber from western Canada. Information Report VP-X-180. Western Forest Products Laboratory, Vancouver, Canada, August.

McGowan, W.M. (1968). Parallel-to-grain tensile properties of visually graded 2 x 6-inch Douglas fir. Information Report VP-X-46. Forest Products Laboratory, Vancouver, B.C., December.

McGowan, W.M. (1971). Parallel-to-grain tensile properties of coast and interior-grown 2 x 6-inch Douglas fir. Information Report VP-X-87. Department of the Environment, Canadian Forest Service, Vancouver, B.C., August.

McGowan, W.M., Rovner, B., and Littleford, T.W. (1977). Parallel-to-grain tensile properties of dimension lumber from several western Canadian species. Information Report VP-X-172. Western Forest Products Laboratory, Vancouver, Canada, October.

Madsen, B. (1972). Duration of load test for wet lumber in bending. Structural Research Series Report No.4, University of British Columbia. Department of Civil Engineering, March.

Madsen, B. (1980). Parameters affecting the efficiency of mechanical grading. Proceedings of IUFRO Conference, Oxford, UK, April, pp.175-192.

Madsen, B., and Knuffel, W. (1980). Investigation of strength-stiffness relationship for South African timbers as it relates to mechanical grading. Proceedings of IUFRO Conference, Oxford, UK, April, p.125-173.

Maki, A.C., and Kuenzi, E.W. (1965). Deflection and streses of tapered wood beams. US Forest Service Research Paper, FPL 34, September.

Norris, C.B. (1962). Strength of orthotropic materials subjected to combined stresses. Report No. 1816. USDA Forest Service, Forest Products Laboratory, May.

Orosz, I. (1968). Some non-destructive parameters for prediction of strength of structural lumber. USDA Forest Service Research Paper, FPL 100, October.

Orosz, I. (1969). Modulus of elasticity and bending strength ratio as indicators of tensile strength of lumber. Journal of Materials, Vol.4, No.4, pp.842-864.

Pearson, R.G. (1965). The establishment of working stresses for groups of species. Division of Forest Products Technological Paper No. 35, CSIRO.

Schniewind, A.P., and Lyon, D.E. (1971). Tensile strength of redwood dimension lumber. I. Relation to grade and working stress. Forest Products Journal, July, Vol.21, No.7, pp.18-27.

Schniewind, A.P., and Lyon, D.E. (1971). Tensile strength of redwood dimension lumber. II. Prediction of strength values. Forest Products Journal, Vol.21, No.8, August, pp.45-55.

Senft, J.F. (1973). Further studies in combined bending and tension strength of structural 2 by 4 lumber. Forest Products Journal, Vol.23, No.10, Oct., pp.36-41.

Senft, J.F., and Suddarth, S.K. (1970). Strength of structural lumber under combined bending and tension loading. Forest Products Journal, Vol.20, No.7, July, pp.17-21.

Timoshenko, S. (1956). Strength of materials. Part I. D. Van Nostrand Company Inc., New York, pp.318-319.

Walters, E.O., and Westbrook, R.F. (1970). Vibration machine grading of southern pine dimension lumber. Forest Products Journal, Vol.20, No.5, May, pp.24-32.

Weibull, W. (1939). A statistical theory of the strength of materials. Swedish Royal Institute of Engineering Research, Stockholm.

Wilson, T.R.C., Carlson, T.A., and Luxford, R.F. (1930). The effect of partial seasoning on the strength of wood. Proceedings of the American Wood Preservers Association, Annual General Meeting, January, pp.349-379.

APPENDIX A
PROPERTIES OF 100 x 50 mm DRY SOUTHERN PINE
(After Doyle and Markwardt 1966, 1967)

Property/material	Mean value (MPa)	5 Percentile (MPa)	Coeff. of var.
COMPRESSION PARALLEL TO THE GRAIN			
- small clear	43	33	0.15
- structural clear	40	31	0.15
- No. 1 grade	37	28	0.15
- No. 2 grade	33	23	0.19
- No. 3 grade	29	19	0.23
BENDING PARALLEL TO THE GRAIN			
- small clear	90	70	0.15
- structural clear	67	47	0.20
- No. 1 grade	62	40	0.24
- No. 2 grade	45	25	0.32
- No. 3 grade	39	18	0.45
TENSION PARALLEL TO THE GRAIN			
- small clear	120	71	0.30
- structural clear	54	29	0.35
- No. 1 grade	39	19	0.39
- No. 2 grade	25	11	0.44
- No. 3 grade	16	7	0.49
MODULUS OF ELASTICITY PARALLEL TO THE GRAIN			
- small clear	12 500	8 090	0.25
- structural clear	12 500	8 090	0.25
- No. 1 grade	12 400	8 310	0.23
- No. 2 grade	10 400	7 280	0.19
- No. 3 grade	9 500	5 610	0.29
MODULUS OF RIGIDITY (G_{LT}, G_{LR})			
- small clear	-	-	-
- structural clear	-	-	-
- No. 1 grade	900	770	0.15
- No. 2 grade	910	660	0.17
- No. 3 grade	920	710	0.14
SHEAR STRENGTH PARALLEL TO THE GRAIN			
-small clear	9.1	-	-
COMPRESSION STRENGTH PERPENDICULAR TO THE GRAIN (2.5 mm deformation)			
- small clear	11.5	-	-

APPENDIX B

STRENGTH THEORY FOR NON-HOMOGENEOUS MATERIALS

Only the essential aspects of this theory will be considered. First it is necessary to recall that for a Weibull distribution of strength R, the probability that R will be greater than x is given by

$$\Pr\ (R > x) = \exp\ (-\ \alpha(x/\bar{R})^m) \tag{B1}$$

where the material parameters α and m are given by

$$\alpha \cong 0.555 + 0.445\ V_R$$
$$m \cong V_R^{-1.09}$$

and V_R is the coefficient of variation of R.

Now consider a structural unit made up of N elements having strengths R_1, R_2, ... R_N and subjected to the stresses σg_1, σg_2, ... σg_N respectively when the nominal stress level in the unit is denoted by σ. The probability of survival by the unit for this stress condition, denoted by $\Pr\ (R_{unit} > \sigma)$, is given by

$$\Pr(R_{unit} > \sigma) = \Pr(R_1 > \sigma g_1).\Pr(R_2 > \sigma g_2)...\Pr(R_N > \sigma g_N) \tag{B2}$$

If it is assumed that all elements have a strength defined by equation (B1), then equation (B2) can be written

$$\Pr(R_{unit} > \sigma) = \exp(-(\alpha\sigma^m/\bar{R}^m)(g_1{}^m + g_2{}^m + \ldots\ g_N{}^m)) \tag{B3}$$

A continuous model of equation (B3) is

$$\Pr(R_{unit} > \sigma) = \exp(-\ [\alpha\sigma^m/\bar{R}^m]\ \int g^m(x,y,z)\ d\phi) \tag{B4}$$

where the parameter ϕ denotes volume and g(x,y,z) is a function of the coordinates x, y and z such that $\sigma g(x,y,z)$ denotes the stress at the point (x,y,z), where σ is the nominal stress level in the member.

By comparison with equation (B1), equation (B4) can be written in terms of $\bar{R}_{unit}$, the mean nominal strength of the structural unit as follows,

$$Pr(\bar{R}_{unit} > \sigma) = \exp(-\alpha\sigma^m/\bar{R}^m_{unit}) \tag{B5}$$

Comparison of equation (B4) and (B5) leads to

$$\bar{R}^m_{unit} = \bar{R}^m / \int g^m(x,y,z)\, d\phi \tag{B6}$$

Thus a computation of the integral $\int g^m(x,y,z)\, d\phi$ for various structural configurations leads to comparative values of the mean strength. It should be noted that since the exponent m remains unchanged, R and R_{unit} are both Weibull distributions with the same coefficient of variation.

STRUCTURAL GRADING OF TIMBER

William G. Keating[1] and Robert H. Leicester[2]

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STRUCTURAL GRADING OF TIMBER

William G. Keating

1. INTRODUCTION

Sound timber grading practice, for either structural or other purposes, lies not in the selection of perfect timber, but on the contrary, it permits the inclusion in each grade of as many defects as is possible without detracting from the suitability of the timber for the purpose for which it is required. Structural timber grading therefore aims to provide the engineer with a degree of confidence so that when specifying a particular grade of a certain species or species group he can expect a consistent minimum quality irrespective of the origin of the material.

The process of structural grading, also known as stress grading, may be defined as the technique by which timber is sorted according to its ability to carry loads.

This process must have a set of rules or criteria for making the sort and a procedure for assigning design properties to each of the grades.

Stress grading is carried out by either one of two techniques, viz. visual or mechanical. These may be further sub-divided depending on the particular approach.

2. VISUAL STRESS GRADING

Visual grading is the oldest stress grading method in use. It is also the simplest and the most widely used. It is based on the premise that the mechanical properties of timber containing certain visible characteristics (defects) differ from those pertaining to timber entirely free from any such characteristics.

In the development of the basic working stresses appropriate to each grade, two different approaches are used. It is worthwhile explaining the procedure in each case to illustrate the cautious attitude adopted.

2.1 Small Clears Approach

Extensive laboratory testing, mainly in USA during the 1920s, indicated that defects of a given size and location cause approximately the same proportional reduction in strength irrespective of species. This fact has permitted the preparation of strength grading rules for timber in which the visible imperfections are limited so that the weakest piece in any grade should have a certain percentage of the strength of a similar piece of clear timber. This is sometimes called the strength ratio. Therefore, for example, a joist with a strength ratio of 60% extreme fibre stress in bending would be expected to have at least 60% of the strength of a corresponding clear piece.

As a first step in the process of the development of basic working stresses, a comparatively large number of small clear specimens of a particular species need to be tested in bending, shear and compression parallel to the grain. The method of testing, sampling and recording of results have been laid down in internationally accepted standards such as ASTM D143. The need to test a large number of specimens for each property is to obtain some indication of the species variability which is then interpreted by normal statistical methods. From the distribution curve obtained the visual practice is to determine firstly the 1% lower probability limit. Taking this limit means that the probability is that 99% of the values will fall above it. Figure 1 illustrates the procedure and indicates that on average the 1% lower probability figure is approximately 70% of the mean, or in statistical terms

$$= \bar{f}_b - 2.33\,\sigma$$

where $\bar{f}_b$ = mean bending strength
σ = standard deviation

$$\text{and } \sigma = \sqrt{([\Sigma(x - \bar{x})^2]/n)}$$

where x = individual test value
$\bar{x}$ = mean of the test values
n = number of tests.

To obtain the basic stress from the 1% probability figure, a reduction factor, k_r, is introduced to allow for long duration loading, accidental overloading, shape and size of specimens, and several other intangibles. Experience over many years has indicated that these may be conveniently combined into one figure. However, it is not necessarily the same figure for every strength property.

The general formula for basic stress in bending

$$= (\bar{f}_b - 2.33\ \sigma)/k_r$$
$$= (\bar{f}_b - 2.33\ \sigma)/2.22$$

for bending stress $k_r = 2.22$.

This then is the basic stress for clear timber free of all strength reducing defects of the particular species tested. In order to arrive at the basic working stress or grade stress, i.e. a stress that can be assigned to structurally graded material for design purposes, it is necessary to apply the strength ratio factor to allow for the loss of strength caused by slope of grain, knots, shakes, checks or other strength-reducing features as detailed in the appropriate visual grading rule. In Figure 1 a strength ratio of 0.48 has been assumed and the foregoing process is summarised as follows for the particular example

Mean	$= \bar{f}_b$	= 54.0 MPa
↓		
1% Lower Probability Limit	$= \bar{f}_b - 2.33\ \sigma$	= 35.0 MPa
↓		
Basic Stress	$= (\bar{f}_b - 2.33\ \sigma)/2.22$	= 15.8 MPa
↓		
Basic Working Stress (Grade Stress)	$= [(\bar{f}_b - 2.33\ \sigma)/2.22]\ 0.48$	= 7.58 MPa

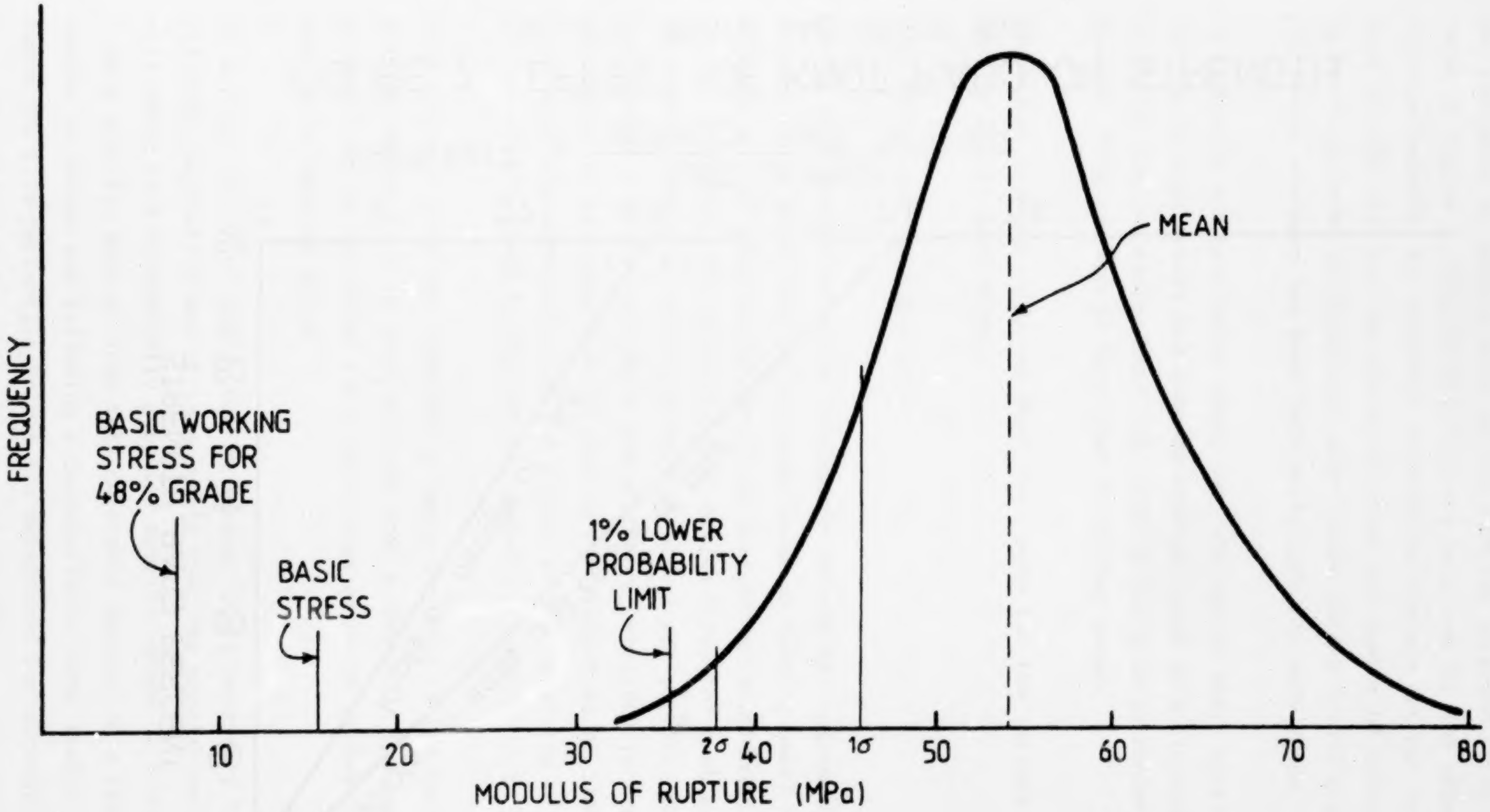

FIGURE 1. DERIVATION OF BASIC WORKING STRESS IN BENDING FOR A 48 PER CENT GRADE OF A TYPICAL SPECIES

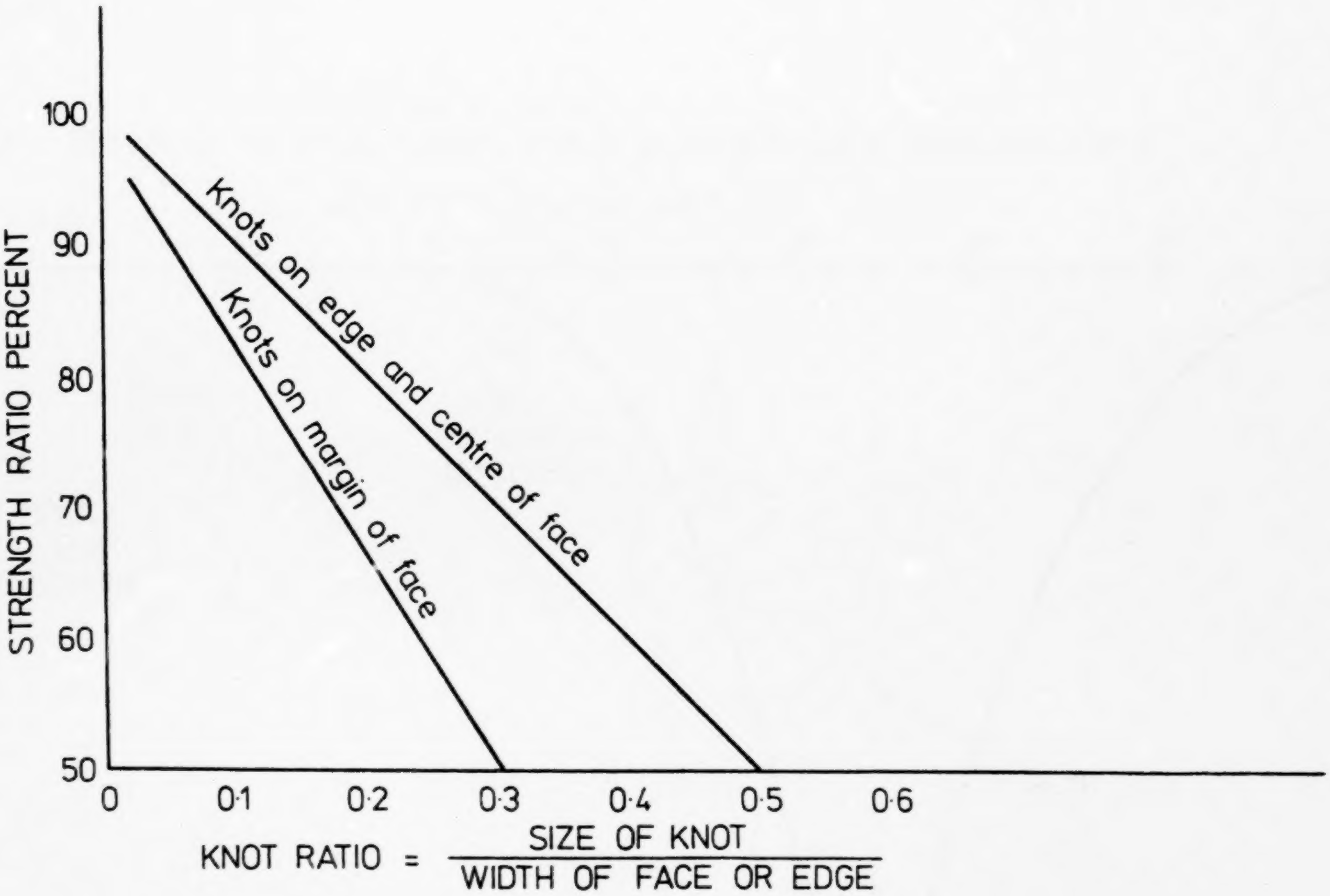

FIGURE 2 : EFFECT OF KNOT RATIO ON STRENGTH

SOURCE : BOOTH AND REECE 1967

The influence of defects such as knots on the strength ratio is illustrated in Figure 2. Similar diagrams may be drawn for the other strength reducing defects. All relevant strength ratios are set out in tabular form in ASTM D245 for the different types and number of defects likely to be encountered in structural size timber during visual grading. This particular standard has been used world wide for many years.

In Australia and many other countries, the set of basic working stresses for the various grades has been determined based on a limited number of groups of species rather than individually for a large number of single species. This process is described in another paper.

The visual grading rules lay down the limits for the various defects appropriate to each grade.

2.2 Structural Size Approach

In the past decade extensive laboratory testing has indicated that the derivation of basic working stresses based on testing small clear specimens and visual grading may result in misleading recommendations for design strengths. The reason is that design strengths must be based on the weaker pieces at the lower tail of structural populations, typically the weakest 1 or 5%; and while the structural characteristics of the average stick of timber are related to the properties of clear wood, the structural characteristics of the timber at the lower end of the distribution is determined by the natural defects occurring in this material.

This approach does not necessarily imply a change in the existing visual grading method, but it does indicate that a change in the basic working stresses may be warranted for a particular grade or grades for a certain species or species group.

In any derivation of basic working stress, it is necessary to define the reference population and it is probable that the population in this case will refer to a commercially supplied specific grade of timber obtained from a specific source. From this reference population a representative sample is chosen and following a comparatively large number of standard tests the five-percentile values are determined for strength and stiffness. From these values, the basic working stresses are obtained by dividing by a load factor that reflects the relationship between three minute loading and long term loading together with a factor that is chosen to match existing practice (factor of safety).

The above technique is in effect a refinement process to produce more efficient designs than those resulting from the small clears approach which must of necessity be conservative. It is apparent that this grade verification process or in-grade testing is expensive and time-consuming and is warranted only when the species or species group comprises a sufficiently large proportion of the total use. (Appendix A sets out the current draft set of rules for the evaluation of graded timber that is being proposed for Australia.)

3. MACHINE STRESS GRADING [1/]

Although visual grading results in greater efficiency in the structural use of timber than if no grading is performed, it suffers nevertheless from a number of disadvantages. Each piece of timber has to be handled and examined on 4 surfaces and having measured the defects it is assumed that pieces of the same species containing defects of the same size are approximately equal in strength. Both these factors lead to high costs. The assumption that pieces containing similar defects are approximately equal in strength implies that all pieces of clear timber of the same species are of equal strength, whereas apparently similar specimens may differ widely in strength due to various factors one of which is density. The presence of knots or sloping grain increases this difficulty of estimating strength since a piece with high inherent strength containing knots may be stronger than a clear piece with low inherent strength. For these reasons visual grading is a somewhat inefficient method and the associated assignment of working stresses must be conservative.

A more direct and accurate approach is to use mechanical grading. This system is based on the fact that there is a direct correlation between the stiffness and ultimate strength of a piece of timber containing defects. This relationship has not always been recognized; in fact, for many years text books stated categorically that while defects affected strength generally in proportion to their size or severity they had no significant effect on stiffness. Pioneer work at the former CSIRO Division of Forest Products on radiata pine showed that such a

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[1/] For a further discussion of machine stress grading see UNIDO document ID/WG.359/3 - The principles involved in stress grading, with special reference to its application in developing countries, 24 May 1984, 64pp., prepared for the Expert Group Meeting on Timber Stress Grading and Strength Grouping, Vienna, 14-17 Dec. 1984 by Mr. C. J. Mettem, TRADA, U.K.

relationship did exist. Further work at CSIRO, the Division of Wood Technology of NSW Forestry Commission, and some overseas laboratories resulted in the development of machines that could utilize this relationship to test timber in a non-destructive fashion to predict its load-carrying capacity. Figure 3 shows the current version of the machine developed by the Division of Wood Technology.

All grading systems are based on the use of predictions to estimate strength properties. In visual grading, for example, the size of visible defects such as knots is used to predict strength. In machine stress grading, the stiffness is used as a predictor. The relationship between the predictor and the mechanical properties of interest is shown by a statistical technique known as a regression. Figure 4 illustrates the use of a regression and the effect of the variability in data on the accuracy of the prediction (Galligan, Snodgrass and Crow 1977). Clearly the tighter the data group around the regression line, the lower the variability and the better the prediction of the strength.

In mechanical grading a non-destructive test is made on each piece by passing it through a machine which continuously applies a small load to it over successive short lengths of the piece. The maximum deflection obtained is recorded and from the known relationship between the modulus of elasticity and the modulus of rupture the ultimate strength of the piece can be predicted.

To illustrate how the system operates, the following example is taken from work carried out in UK by Sunley, Curry and Hudson and reported in 1962 and 1964 (Booth and Reece 1967). A typical relationship between the modulus of elasticity and modulus of rupture is shown in Figure 5. The variability of the material is then taken into account by calculating confidence lines above and below the regression lines such that 98% of the results lie between these lines. The lower confidence line which is shown in Figure 5 is such that only 1 in 100 results will be below the line. The confidence lines are determined by the usual statistical technique of drawing lines, parallel to the regression line and at a vertical distance away of ± 2.33 times the standard error of estimate of the modulus of rupture. By applying factors to allow for long term loading and a factor of safety the recommended grade (working) stress line is shown.

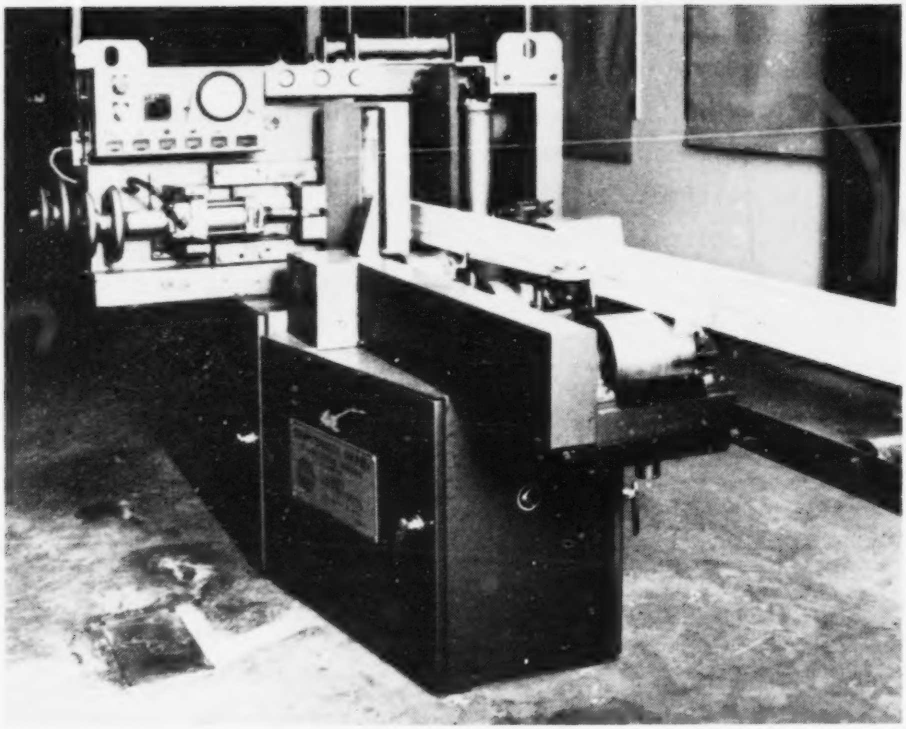


Figure 3 **Computermatic stress grading machine. (Photo by courtesy Forestry Commission of N.S.W.).**

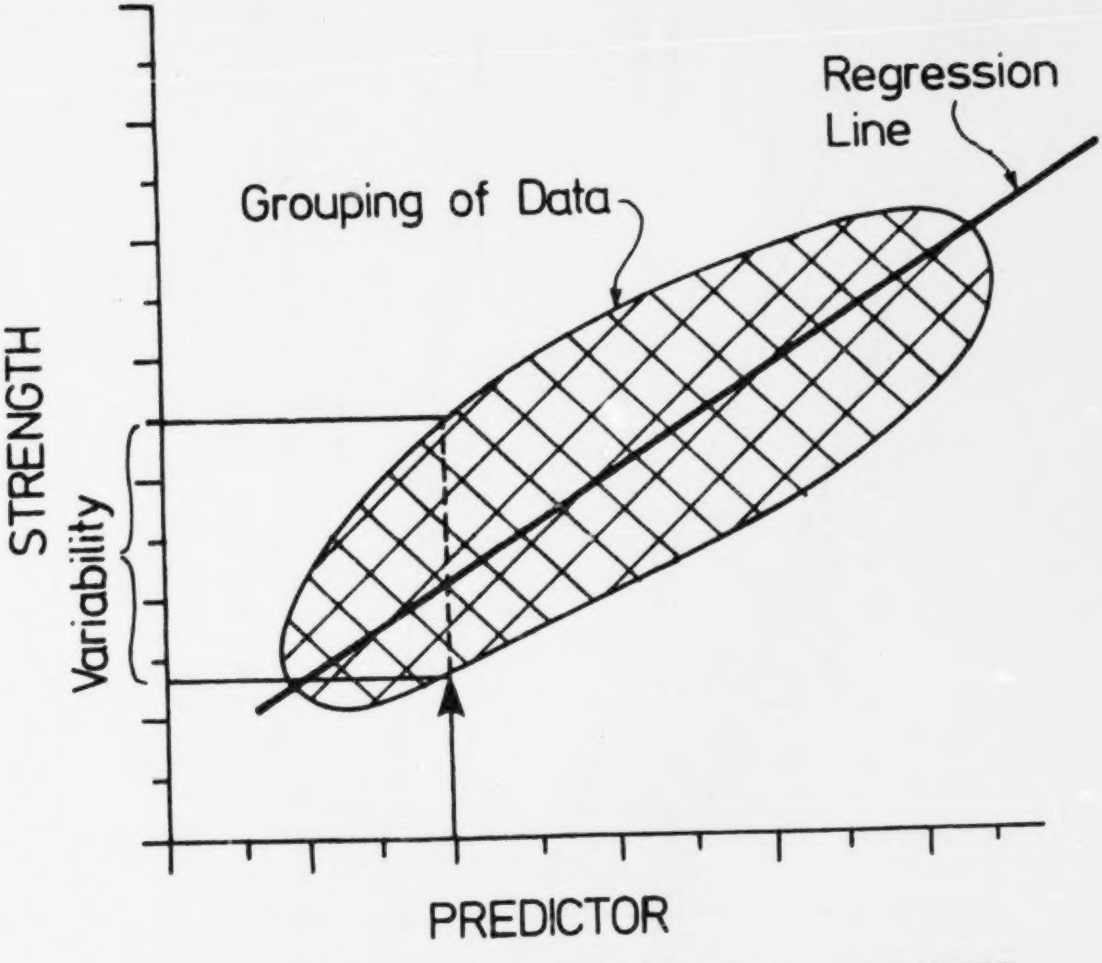


FIGURE 4. TYPICAL RELATIONSHIP BETWEEN A PREDICTOR AND STRENGTH OF TIMBER.

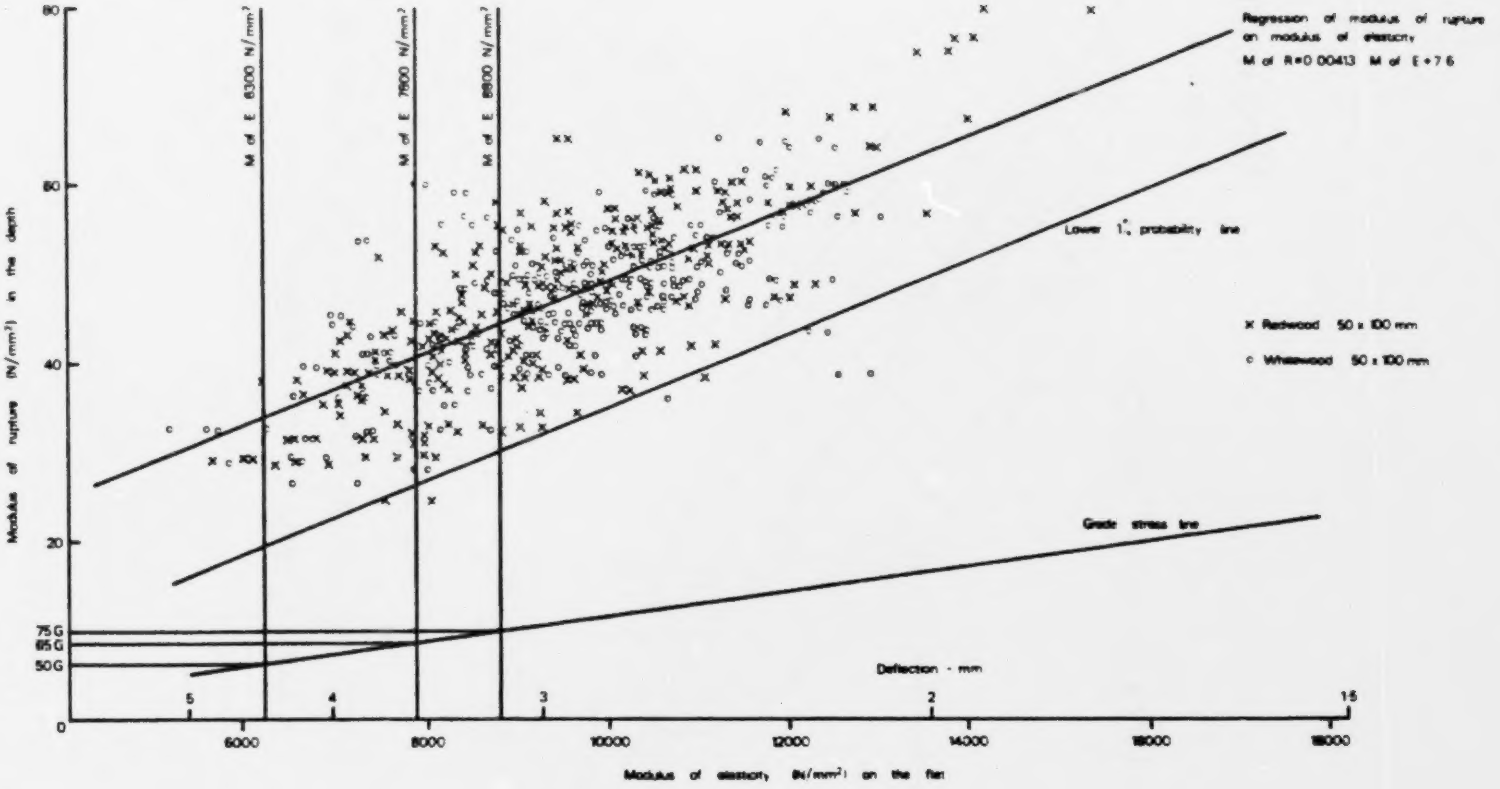


Figure 5. Regression, lower 1% and grade stress lines: European redwood and whitewood

Table 1 shows the results of visual and mechanical grading for Baltic redwood (Pinus sylvestris L.) and whitewood (Picea abies (L.) Karst.) as obtained by Sunley and Hudson (1964). Thus mechanical grading gave either a larger yield of material in the higher grades or higher working stresses for similar grade yields, as obtained by visual methods.

TABLE 1
A COMPARISON OF THE RESULTS OF VISUALLY AND MECHANICALLY GRADED 2" x 4" BALTIC REDWOOD AND WHITEWOOD
(Source: Booth and Reece 1967)

Visual grades	Visual grade stresses (18% moisture content) (lbf/in²)	Visual grade yields (%)	Yields of mechanical grades for stresses in column 2 (%)	Grade stresses for mechanically graded timber to give same yields as column 3 (lbf/in²)
1	2	3	4	5
75 Grade	1450	35	68.0	1700
65 Grade	1250	33	18.5	1450
50 Grade	950	23	12.0	1150
Reject		9	1.5	

The machine produced and available in Australia is marketed under the trade name 'Computermatic'. Pieces of timber are fed into the machine on edge in a longitudinal direction. The individual piece is continually deflected in its narrow dimension by a given load. The amount of deflection caused by this load is measured on 150 mm intervals throughout the length of the piece. Measurements are grouped into 1 of 5 classes. A colour mark can be sprayed on the pieces at each 150 mm interval that corresponds to the grade class. As the piece leaves the machine the lowest grade rating is computed and a paint spray corresponding to that low point rating is sprayed. Figure 6 shows the operation of the machine.

The development of this machine was the result of a cooperative venture between the NSW Forestry Commission and the Plessey Company. It has met with considerable success in Europe where there are now approximately 100 stress-grading machines of this type in use and considerable interest is currently being shown in USA for use in lower production mills (Serry 1978).

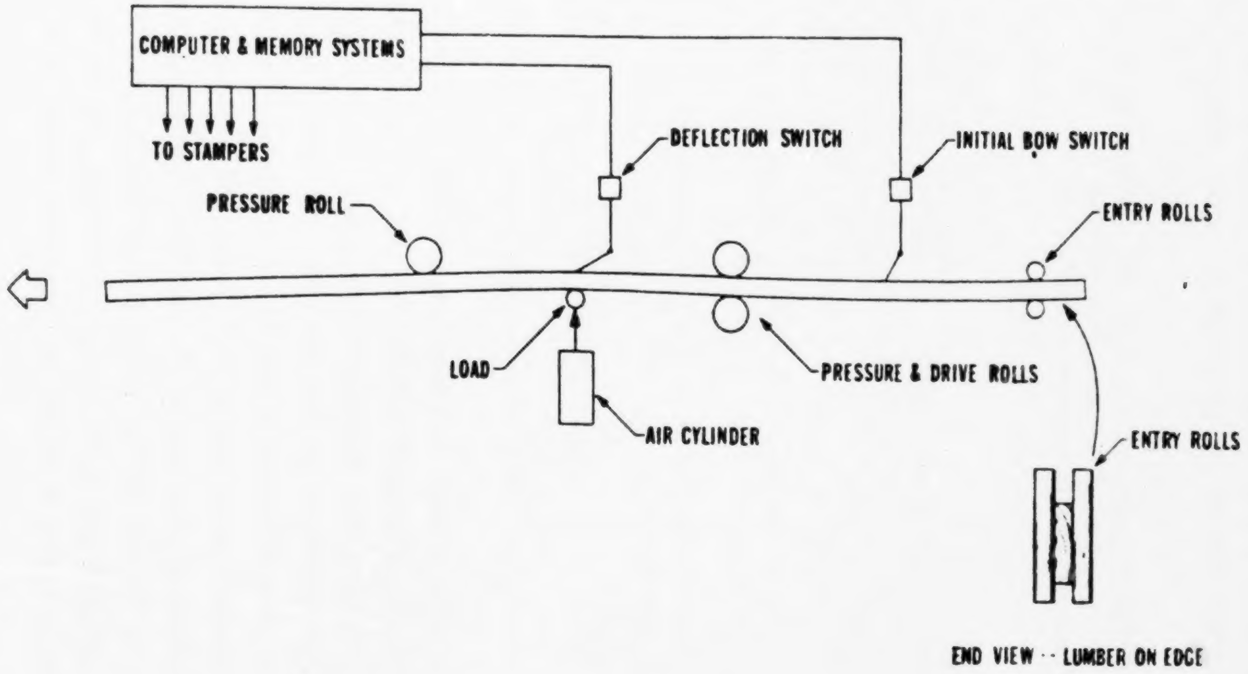

COMPUTER & MEMORY SYSTEMS
TO STAMPERS
DEFLECTION SWITCH
INITIAL BOW SWITCH
PRESSURE ROLL
ENTRY ROLLS
LOAD
PRESSURE & DRIVE ROLLS
AIR CYLINDER
ENTRY ROLLS
END VIEW -- LUMBER ON EDGE

Figure 6. --Illustration of the Computermatic machine.

There are several other machines now on the market and their operation has been summarised by Mettem (1982) as follows:

Cook Bolinder SG-AF stress grading machine

This machine, which was introduced in 1979, receives the timber on edge, and deflects it laterally over a 0.9 m span, in the same configuration as the Computermatic. A high level of accuracy is claimed, probably justly, by using the principle of constant deflection, load sensing. This is a reversal of the Computermatic method which loads a given timber and size to a constant stress, and senses the ensuing deflections, which vary along the piece.

The timber is deflected in opposite directions in two separate passes through the this machine. A computerised sensing mechanism reads off the variable force caused by the fixed deflection, the force being detected by a load cell. During the second pass the computer provides a continuous read-out of the mean values. Dye marks and stamp marks are applied to indicate grade.

By using separate passes the interference effects associated with double bending are eliminated. By using a constant deflection it is possible to avoid errors in load-deflection measurement caused by timber vibrations.

The Raute Timgrader stress grading machine

The technical literature describing this machine dates from 1978, the approximate date of its introduction, although work on prototypes had started earlier.

Timber is deflected horizontally, whilst passing through on edge, in common with both machines described above. The Timgrader is a fixed deflection, variable load machine, like the Cook Bolinder type. A difference in principle is that in this machine the method of allowing for bow is to bend the timber successively in opposite directions in a single pass. The variable forces required to provide fixed deflections in this manner are sensed and averaged electronically. The measuring frequency is synchronised with the feed speed of the timber at about 100 mm intervals along the length passing through.

The machine embodies quite a substantial electronic system including a process control microcomputer and a unit which can transmit grading information to a sorting machine.

In deriving settings for this machine, it is necessary to calculate allowances for interaction between a low stiffness portion of the length (due perhaps to a gross defect) passing one defletion station whilst material of a different quality (another part of the length) passes the second station.

The Sontrin timber selector

This machine has not been approved for general purpose stress grading to BS 4978 under the Kitemark system because of its insensitivity to localised gross defects, particularly away from the centre of the span. These may be of importance in structural members with built-in or partially fixed ends such as rafters or trussed rafters. The timber selector is of value however in grading simply supported bending members such as joists. It may be used either as an adjunct to visual grading, taking advantage of the machine's ability to eliminate pieces of low gross stiffness, or in conjunction with a setting procedure which has been devised to weight the required apparent modulus in the settings according to the distance of the suspected defect from the lateral centre line of the piece.

The machine subjects a piece of timber to a predetermined central load and senses the ensuing deflection, giving a simple pass/fail indication for a particular setting. The material is deflected in the depthwise direction, not laterally as in the machines described above. Variable spans are used, these being related to the intended end use of the piece and ranging from 2.4 m to 5.4 m.

The Plan-Sell, Innotec Oy Finnograder device

This third generation device uses non-contacting measurement methods, and consequently it is possibly not strictly correct to refer to it as a machine. The manufacturing company was established in 1973 and was taken over as a subsidiary of Plan-Sell Oy in 1978. The Finnograder has been assessed at the Finnish VTT Technical Research Centre.

The Innotec instruments, which include also an on-line moisture meter and an automatic control system for edging, perform measurements by various kinds of electromagnetic radiation. The Finnograder is claimed to detect density, knottiness, slope of grain and moisture content. By relations between these features and tested strengths, performance can be predicted in bending about either axis or in tension.

The ISO-GreComat grader

The principle of this machine, which has been assessed at the Otto-Graf Institute in Stuttgart during 1980 and 1981, is to grade timber by measuring its density through use of isotopes. Knot area ratios are assessed by determining ratios between local and general density, and allowances are made for influences of moisture content and dimensional variations.

Kennedy (1982) has described the main machine used in Canada in the following manner:

MSR lumber in Canada is produced exclusively by passing kiln-dried lumber through a 'Continuous Lumber Tester', or CLT-1, marketed by Metriguard Inc. of Pullman, Washington. This machine deflects lumber over a series of 4 foot spans in both vertical directions as it passes through the machine at rates up to 1200 lineal feet per minute. The load required to achieve a given deflection is recorded in the memory of the machine, to calculate average and minimum E values, which in turn are correlated with F_b values through previously established regression equations. The tentatively machine-rated lumber is still visually graded and subjected to a 'visual override', which culls those pieces with edge knots, which would severely limit the performance of the lumber when used on edge, but which would not be detected as the lumber moved through the machine on its wide face.

Mills producing MSR lumber must have the approval of the Canadian Lumber Standards Accreditation Board, to whom a written plant standard must be submitted. The standard describes the operation of their grading machine, and the qualifications of its operators and quality control personnel. The trade association responsible for grade marking grants initial approval for MSR production after sampling and independent testing of grade-marked material to insure accuracy of the grading machine. Continuous quality control involves daily mill tests of production, cumulative record keeping, maintenance of grading and static testing machines, and periodic visits by agency supervisors. Product require-ments and producer responsibilities are outlined in a recently published specification.

4. PROOF GRADING

Proof grading, which could be classed as a particular type of machine grading, is finding increasing application for many situations. The following paper by R H Leicester describes the technique with some additional information in Appendix E.

5. REFERENCES

American Society of Testing and Materials (1982). ASTM D143-52. Standard Methods of Testing Small Clear Specimens of Timber.

American Society of Testing and Materials (1982). ASTM D245-81 Standard Methods for Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber.

Booth, L.G., and Reece, P.O. (1967). The Structural Use of Timber. A Commentary on the British Stanard Code of Practice CP112. Spon Ltd, London, p.287.

Ethington, R.L., and Fluscher, H.O. (1973). A Perspective on Structural Lumber Grading. Forest Products Journal, 23(9):54-55.

Galligan, W.L., Snodgrass, D.V., and Crow, G.W. (1977). Machine Stress Rating. Practical Concerns for Lumber Producers. Technical Report FPL 7, United States Department of Agriculture, Madison, Wisconsin, p.75.

Kennedy, R.W. (1982). Stress Grading of Canadian Softwood Lumber. Institute of Wood Science (Australian Branch), Melbourne.

Leicester, R.H. (1981). Structural Utilisation of Timber. Topic H8, 20th Forest Products Research Conference, Melbourne.

Madsen, B. (1978). In-grade Testing, Problem Analysis. Forest Products Journal 28(4), pp.42-50.

Mettem, C.J. (1982). The Principles Involved in Stress Grading, with Special Reference to its Application in Developing Countries. UNIDO Paper ID/WG 359/3.

Serry, J. (1978). Stress Grading - A Year of Consolidation. Timber Trades Journal. 1978 Special Issue, London.

Sunley, J.G., and Hudson, W.M. (1964). Machine Grading of Lumber in Britain. Forest Products Journal 14, pp.155-158.

PROOF GRADING OF TIMBER

Robert H. Leicester

It has recently become mandatory in most areas of Australia, for all timber sold for structural purposes, including that to be used in house framing, to be stress graded. This ruling has created difficulties in the supply of timber, because the two methods that are available for the grading of timber are not completely suitable for use in the small mills that produce much of the structural material.

One of the two available methods of grading, the visual grading method, is based on the correlation between visual appearance and strength of timber and is very difficult to apply in the confused and untidy environment of the typical small mill. The other method available for grading is machine stress grading which is based on the correlation between local stiffness and strength of timber. For the small mill, the drawbacks of this method include the expensive cost of the machine and ancillary equipment, usually in excess of $80,000, and the requirement of a sophisticated technological back-up. It was to overcome these difficulties that the concept of proof grading, a method of grading through proof testing, was proposed.

The essence of the proof grading proposal is to allow any given stick of timber to qualify for a particular stress grade if it first demonstrates the ability to sustain a proof bending stress specified for that particular stress grade. The proof bending stress is applied by a proof testing machine. A schematic of this procedure in its simplest form is shown in Figure 1a.

There are several features of proof grading that are obviously well suited for application in a small mill. The method does not require a high level of technology and the machine to do the grading is both simple and inexpensive. It has been estimated that a machine for continuous on-line testing should cost considerably less than $10,000.

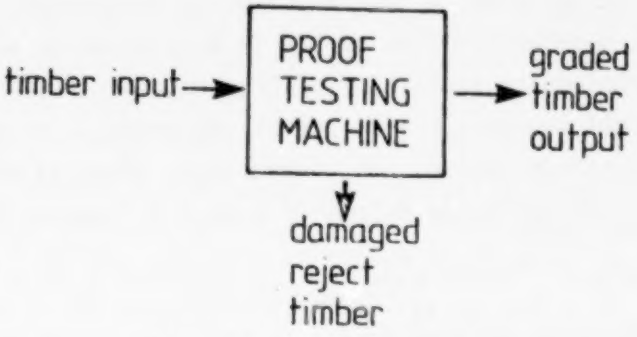
However, even when applied outside the small mill situation, proof grading has some advantages over other grading methods. In contrast to these other methods, proof grading can assess the strength of very localized defects such as those associated with compression shakes or fabricated finger joints. It can also be applied to mills that handle mixed unidentified species, a situation that occurs in many areas of Australia. Finally, by changing the proportion of reject timber produced through changing the proof stress applied, the stress grade of a specific population of timber can be varied, and consequently there is some flexibility available in adopting the output of a mill to meet varying market situations.

A commercial disadvantage that arises in the application of proof grading to the output of a large mill is the fact that the output of a proof testing machine is timber of only one stress grade. To some extent this disadvantage can be minimized, and more than one stress grade produced if use is made of some rough initial presorting technique, possibly through a visual method or through the use of a simple stiffness sorting machine. An outline of such a scheme is shown in Figure 1b.

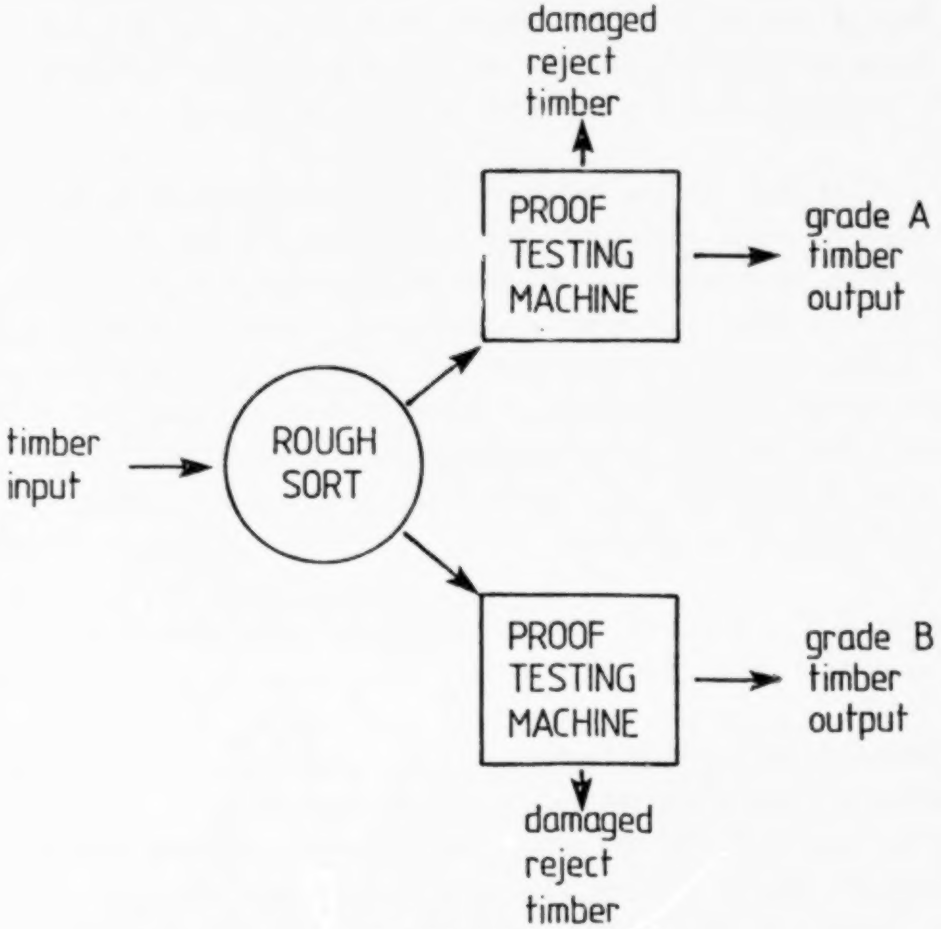
In order to investigate the feasibility of proof grading, and in particular to study the numerous commercial ramifications of its application as outlined above, an extensive set of laboratory and field studies have been made. So as to cover a representative range of Australian timbers and market situations, these studies have been carried out in cooperation with the Radiata Pine Association of Australia based in Adelaide, the Queensland Department of Forestry and the Timber Research and Development Advisory Council, both based in Brisbane, and the Timber Promotion Council based in Melbourne.

Laboratory and mill studies have been undertaken to investigate the following aspects of proof grading timber:

(a) The damage caused by proof testing.
(b) The effect of operator error in single pass grading.
(c) The relationship between the proof load in bending, and the tension strength and modulus of elasticity of the graded material.
(d) The structural reliability required in the basic design stresses and modulus of elasticity of proof graded material.
(e) Quality control of grading operations.
(f) The calibration of grading machines.
(g) The commercial viability of proof grading.


timber input
PROOF TESTING MACHINE
graded timber output
damaged reject timber

(a) For Small Mill


damaged reject timber
PROOF TESTING MACHINE
grade A timber output
timber input
ROUGH SORT
PROOF TESTING MACHINE
grade B timber output
damaged reject timber

(b) For Large Mill

Figure 1 Sequence of operations for proof grading procedure

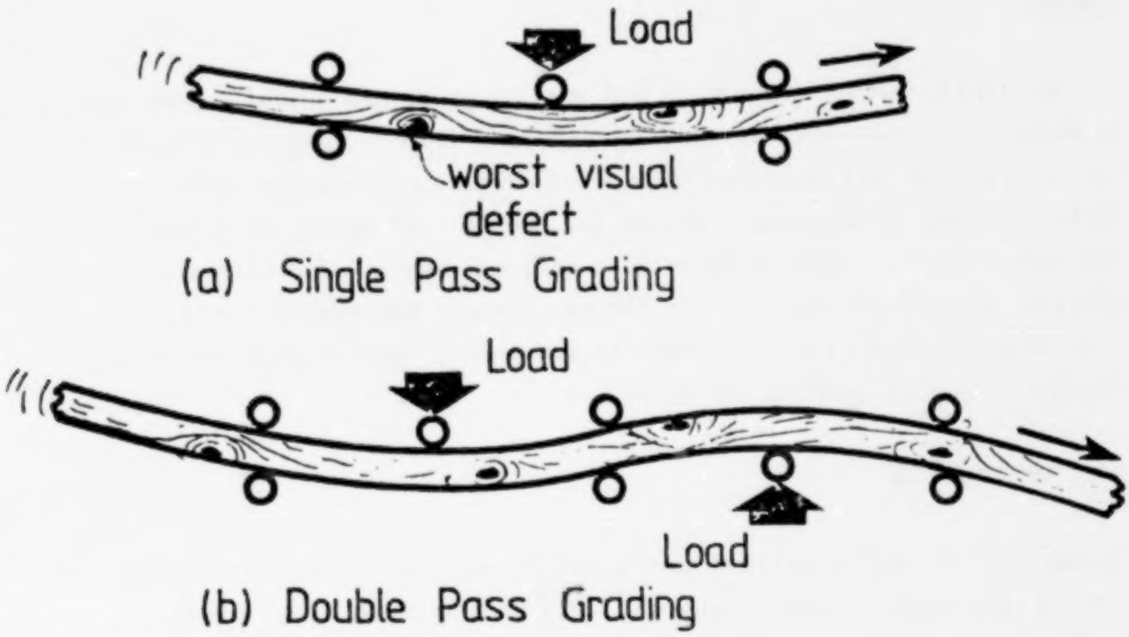

Load
worst visual defect
(a) Single Pass Grading
Load
Load
(b) Double Pass Grading

Figure 2 Schematic illustration of single and double pass grading

As can be seen from the foregoing discussion, there are several ways of grading structural timber and the number is likely to increase in the future. The following paper, also by R H Leicester, is presented as an aid in the decision-making processes.

MODEL OF THE TIMBER GRADING PROCESS

Robert H. Leicester

SUMMARY

In the future, both the quantity and quality of stress graded lumber may be expected to increase. In addition, there will be a need for a great variety of grading procedures because of differences between mills in number of species processed, volume throughput, and access to finance and technology. In order to provide a rational basis for developing suitable grading schemes for the future, a model for evaluating the cost-effectiveness of such schemes is presented. Some prognostications for the future of grading are given.

1. INTRODUCTION

Stress grading may be defined as the act of sorting lumber into groups, each having a set of specified structural design properties. These design properties may be conveniently subdivided into strength and stiffness types. To appreciate the following discussion, it is important to realize the significance of reliability in structural design, particularly with respect to matters concerning strength. Because of this, it is incorrect to associate design properties with individual sticks of timber. Rather, these properties refer to statistically defined parameters of all lumber contained within a given stress grade.

Stress grading is usually specified as mandatory for timber to be used in engineered structures (e.g. roof trusses). Stress grading is also frequently specified for semi-engineered construction (e.g. timber framed houses). At the moment the attitude of the timber industry to stress grading may be best described as one of mixed enthusiasm, with some segments maintaining a high level of grading and quality control, while others virtually ignore mandatory grading specifications. However, the indications are that grading is becoming increasingly appreciated, and consequently future grading will probably be completely different from that of the present.

There are many reasons why both the quantity and quality of stress grading may be expected to increase in future. The most obvious reason is the growing appreciation of the economies to be obtained through grading. For example, in Australia the strongest grades of timber used are about 10 times as strong as the weakest ones (Standards Association of Australia 1975, 1979a); obviously in such a situation there is a strong economic incentive to differentiate between the various structural grades of timber, even when the end use is semi-engineered house construction. The same problem of utilizing a wide range of species exists in most countries with tropical forests (Pong Sono 1974, Espiloy 1978, Wong and Wong 1980), and so the use of grading may be expected to become more widespread as timber production in these countries increases.

The advantages to be obtained from stress grading are considerably enhanced if it is used within a strength grouping format (Leicester 1980). In this format, the design properties of all grades of timber, perhaps numbering several hundred in any one particular country, are replaced by a small set of hypothetical grades. Obviously this will simplify the design of timber structures and will be particularly useful in the utilization of lesser-known and less common species. Further benefits are obtained if there is international acceptance of a common strength grouping system. Such an acceptance will assist not only in the trade of structural timber, but also in the transfer of technology software, such as standard designs, from one country to another.

With the increased acceptance of stress grading, it may be expected that there will be a corresponding increase in the types of situations in which it is used. For example, the timber throughput, range of sizes, and numbers of species handled will vary considerably from one mill to another. Similarly there will be large differences between mills with respect to the financial capital and technology available. Because of this great range of situations, it can be expected that a great variety of grading methods will be employed in order to maximize the benefits that are to be obtained through stress grading.

In order to discuss, compare and devise grading methods, it is essential to have universal models of the grading procedure. Suitable models for this do not exist at present, and the development of such models will probably be a focus of research interest in the near future. The following discussion will be based on a simple model that contains the essential features of grading methods.

2. A MODEL OF STRESS GRADING

2.1 Concepts

Figure 1 is a schematic illustration of the basic elements of grading. The essential feature of the procedure is the sorting of a parent population into various grades, such as the grades 1, 2 and 3 shown.

The parent population may be chosen in any way desired. However, once chosen, it must be rigorously defined and observed. Typically, it is a specified segment of the timber fed into the mills in a suitably chosen geographic region over a period of about ten years. The population may be an identified species, a defined mixture of three to six species, or even an unspecified mixture of many unidentified species.

Evaluation tests are used to provide information for setting the parameters in the chosen grading procedure. The statistical characteristics of the structural lumber used for these tests must be representative of those of the parent population. There is a wide range in the types of evaluation tests that are undertaken. As the amount of testing increases so too does the reliability of the knowledge on lumber properties, and accordingly the grading can be made more efficient. The following are typical examples of evaluation tests, presented in order of increasing efficiency and cost, that may be undertaken for a population that has not previously been stress graded:

(a) No tests
(b) Measurement of population density only (Standards Association of Australia 1979b)
(c) Measurement of bending and compression strength of 10-50 small clear timber specimens (Standards Association of Australia 1979b)
(d) Measurement of bending strength and stiffness of 30-200 sticks of structural lumber (American Society for Testing and Materials 1980)
(e) Measurement of bending strength, tension strength and stiffness of 1000-10,000 sticks of structural lumber.

The grade verification process, sometimes referred to as 'in-grade' testing (Madsen 1978), is expensive and time-consuming, and hence it is not always applied. The most common reason for its application is to resolve doubts that a particular grading procedure is leading to the stress grade that is claimed. In so doing, the process plays an important role in efforts to mitigate the chaos that tends to arise from the conflicts inherent in marketing competing species or competing grading systems.

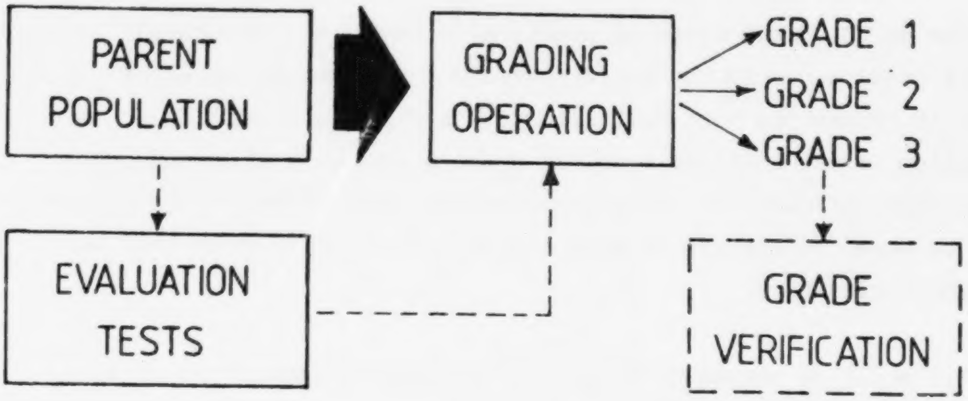

PARENT POPULATION
GRADING OPERATION
GRADE 1
GRADE 2
GRADE 3
EVALUATION TESTS
GRADE VERIFICATION

Figure 1 Basic elements of grading

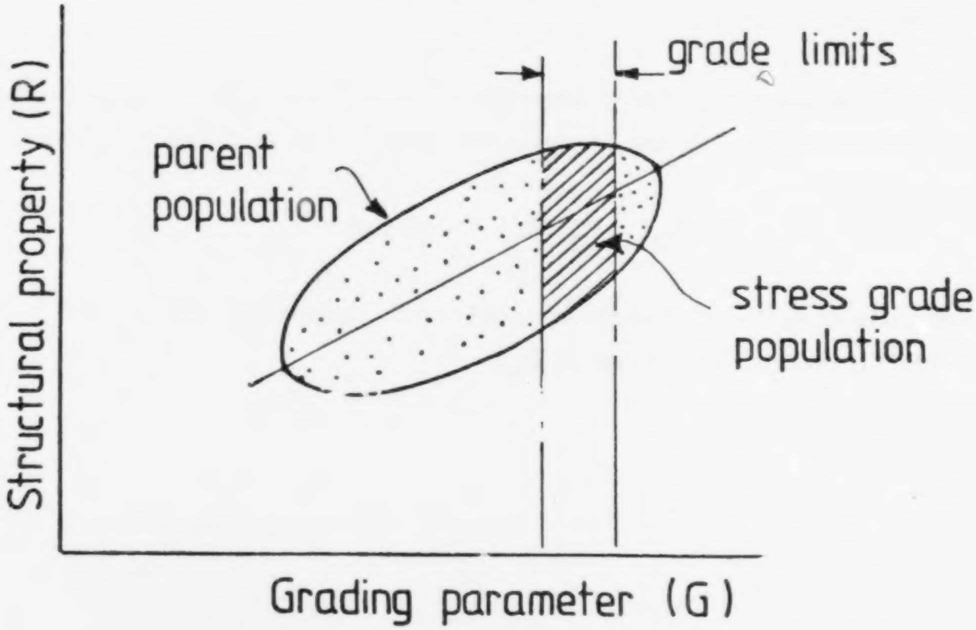

grade limits
parent population
stress grade population
Structural property (R)
Grading parameter (G)

Figure 2 The grading operation

Many structural properties, such as strength, cannot be measured directly during grading. Hence the grading operation, illustrated diagrammatically in Figure 2, is based on the correlation that exists between the structural property of interest, to be denoted by R, and a grading parameter, to be denoted by G.

2.2 Statistical Characteristics of Grading

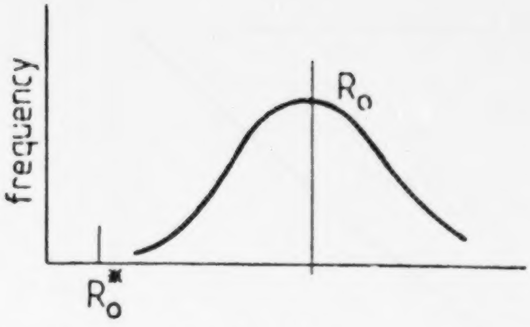
Because of the great range of practical situations encountered, no single grading system can be recommended for universal application. In order to choose the most cost-effective system for a particular situation, a mathematical model of grading is required in order to assess the influence of various parameters. The following discussion will be based on the use of such a model, details of which will be presented elsewhere.

In this model of stress grading, the statistical properties of the parent population, denoted by R_o, are distributed among three populations, denoted by R_1, R_2 and R_3 in Figure 3b, comprising in total roughly three-quarters of the total parent population. The design values of R for the three stress grades are R_1*, R_2* and R_3* and are also shown in Figure 3b.

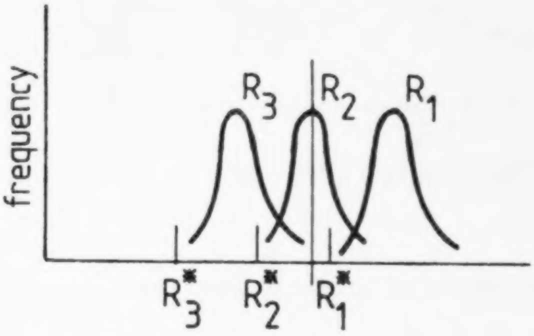
The ratio between the design strengths of these populations is of some relevance to marketing the structural lumber, and is roughly given by

$$R_1^*/R_2^* \cong R_2^*/R_3^* \cong \exp(0.6\ r\ V) \qquad (1)$$

where r denotes the correlation coefficient between G and R_o, and V is the coefficient of variation of R_o.



(a) Parent Population



(b) Graded Population

Figure 3   Distribution of structural property

A measure of the average effective increase in design property due to the grading operation is given by the ratio R_2^*/R_0^*, which may be roughly computed from

$$R_2^*/R_0^* \cong \exp \{0.6\, \beta\, V\, [1 - \sqrt{1 - 0.95r^2}] - (V/\sqrt{N})\sqrt{1 + 0.2\, \beta^2}\} \qquad (2)$$

where R_2^* is the design value of R_2 based on an evaluation with a sample of size N, R_0^* is the nominal ideal design value of the parent population based on an evaluation with an infinite sample, and β is a safety index (Sexsmith and Fox 1978, Ellingwood et al. 1980). Typical parameters for strength properties are $V = 0.5$, $\beta = 3$ and for stiffness properties $V = 0.2$, $\beta = 1$. The effects of the correlation coefficient r and sample size N are shown in Figures 4, 5 and 6.

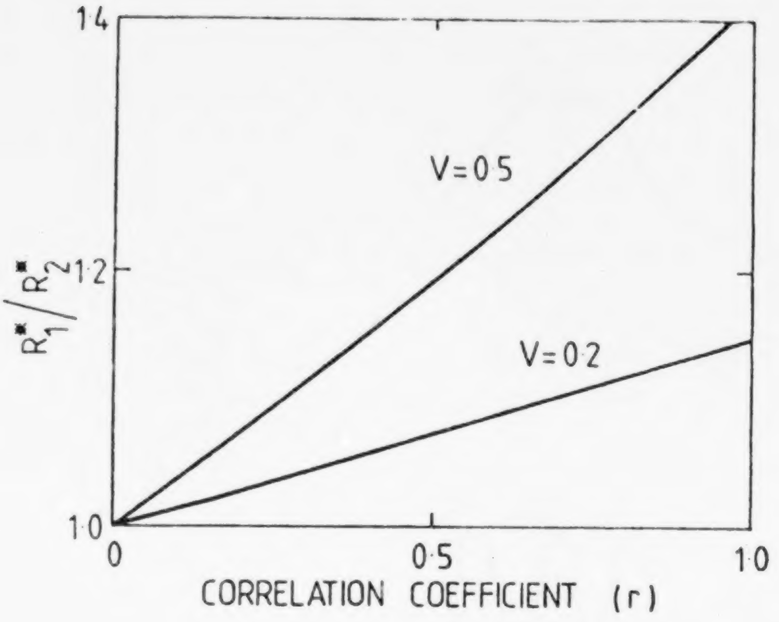


Figure 4 Separation of grade properties

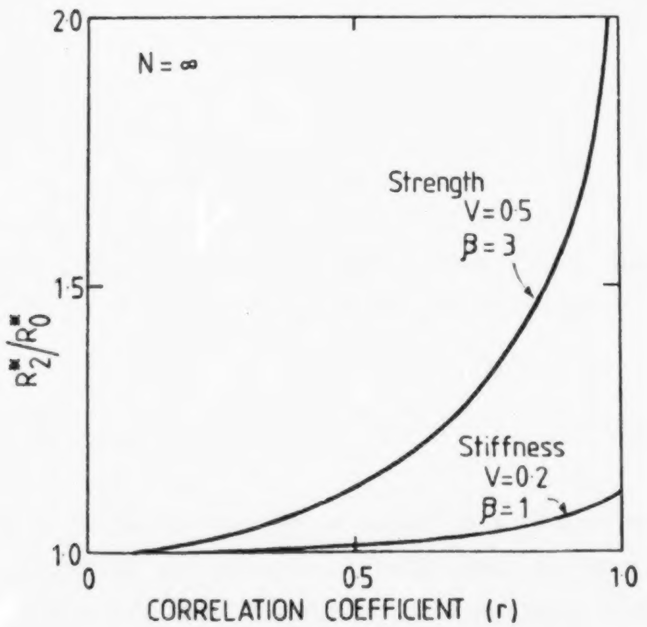


Figure 5 Increase in design values

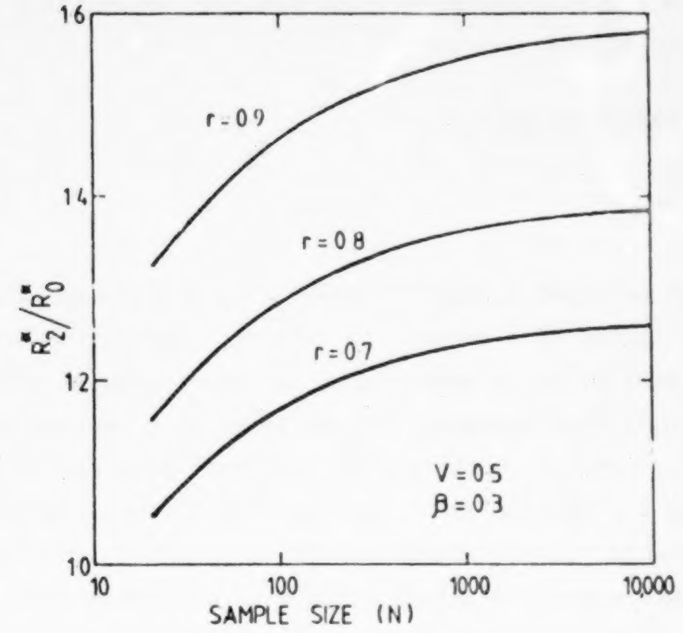


Figure 6 Effect of sample size in evaluating tests

After evaluating R_2*/R_0* from equation (2), the cost C_G of building with graded timber, relative to C_U, the nominal cost of building with well-sampled but ungraded timber, may be obtained from

$$C_G/C_U \cong (1 + C_g)(R_o*/R_2*)^{0.8} \qquad (3)$$

where C_g denotes the cost of grading relative to the initial cost of ungraded timber.

For practical purposes, two refinements of equation (2) may be required. To account for inaccuracies by the grader, the parameter r is replaced by rr_e, where r_e denotes the correlation coefficient between the true grading parameter and the apparent value measured by the grader. In addition, a major modification to equation (2) is required if the evaluation procedure does not involve tests on structural lumber of the parent population concerned. In this situation, the grading operation is based in part on tests with structural lumber of other populations. As a result, it can be shown that the ratio R_2*/R_o* depends also on V_λ, the coefficient of variation of λ, and r_λ, the correlation coefficient of λ with G, where A is a population made up of the mean values of R from various parent populations.

3. SUMMARY OF GRADING METHODS

3.1 Visual Grading

This is the oldest and most widespread grading method in use. The grading parameter is the size of visible defects, used either on its own or combined with some other parameter such as clear material strength. The quantification of this parameter is specified in a variety of ways. Some are relatively complex, such as with the 'knot area ratio' method (British Standards Institution 1973) that notionally involves an X-ray picture of the defects. Other methods can be very simple, such as that specified for heart-in studs of Pinus radiata (Standards Association of Australia 1979c), which considers only the defect size along the edge of the lumber.

3.2 Mechanical Stress Grading

The grading parameter used in this method is the modulus of elasticity. The particular modulus measured may be E_{min}, the minimum local modulus along the length of the stick of lumber, E_{av}, the modulus averaged along the stick, or a combination of the two (Madsen and Knuffel 1980).

Most of the available commercial machines make static measurements of E_{min}. Some, such as the Plessey Computermatic (Anon. 1972), are expensive continuous-measurement machines, capable of high throughput rates. Others, such as the TRU Timber Grader (Bryant 1978), are low cost, low throughput machines which make spot measurements of the modulus. Machines that make dynamic measurements of the modulus usually measure E_{av} (Schniewind and Lyon 1971, Walters and Westbrook 1970).

3.3 Electronic Grading

High-speed non-contacting electronic graders have recently become available. One example is the Finnograder (Innotec Oy 1980). These measure simultaneously several lumber characteristics such as density, moisture content, knot size and location, and slope of grain. Through the use of microcomputers, very complex and hence potentially very efficient grading parameters may be employed.

3.4 Proof Grading

Proof grading (Anon. 1981, Leicester 1979b) has recently been introduced in Australia. It is illustrated schematically in Figure 7. The lumber population is first sorted roughly, usually by rapid visual scanning, into two or more grades. Each grade of lumber is then passed through a continuous proof testing machine which will cull out the exceptionally weak pieces of timber. In a typical grading operation, the proof load is chosen so that about 1-3% of the timber fails and is rejected. Thus the grading parameter in this procedure is the bending strength at the tail of the graded population of lumber.

The proof grading system was developed for application in a situation where there is little available capital and technology.

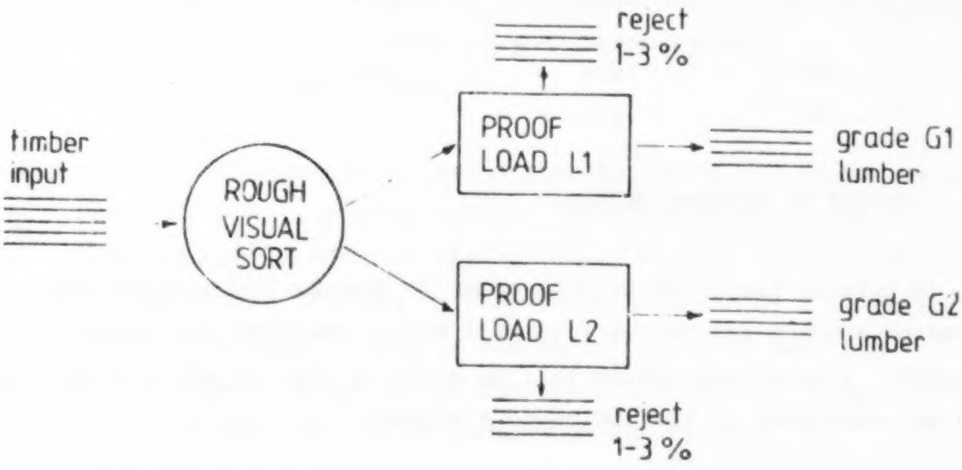


Figure 7 The proof grading procedure

4. COST OF GRADING

Apart from one example for a high speed mechanical stress grader (Ince 1979), there is little detailed published information on the cost of grading timber. The cost of a grading machine is about $US100,000 for one with a throughput rate of 200 m/min, and $US10,000 for one with a throughput rate of 20 m/min. Typically two men are required to run a grading operation.

Table 1 gives a rough estimate of some cost components for machine grading in Australia. The costs associated with visual grading are similar to those for a machine with a throughput of 20 m/min. To compute realistically the added cost of grading, additional information is required on matters such as the amortization period, discounted cash flow and inflation rates, and the number of evaluation tests proposed. Typical values of the relative grading cost parameter C_g are in the range 1-10%, with the smaller costs being associated with higher throughput grading machines.

TABLE 1
SOME COST COMPONENTS OF MACHINE GRADING

Machine throughput rate (m/min)	(Cost x 100)/(annual retail value for one year's throughput of timber graded by the machine)		
	Initial machine cost	Annual running costs	Evaluation tests on 10,000 sticks
200	0.6	0.6	0.6
20	0.6	3.0	6.0

5. CHOICE OF GRADING METHOD

It is outside the scope of this paper to discuss the application of grading schemes for the various mill situations that are possible. However, a brief discussion will be given of various matters that need to be considered in the choice of a scheme.

Equations (2) and (3) provide a useful basis for decisions related to grading timber. To use these equations, the coefficients V and r of the parent population must be estimated. There is enough information available in the published literature to provide rough estimates of these parameters. Some of this information is given in the literature cited at the end of this paper (Curry and Tory 1976, Dawe 1964, Gerhards and Ethington 1974, Leicester 1979a, Madsen and Knuffel 1980, Orosz 1969, Schniewind and Lyon 1971). For strength properties $V = 0.3$ to 0.6 and typical values of r for various grading parameters are as follows:

visual strength ratio	$r = 0.6$
electronic strength ratio	$r = 0.7$
modulus $E_{min.}$	$r = 0.85$
modulus E_{av}	$r = 0.5$

The above are only indicative, and the true values of r can vary considerably from one parent population to another. For example, for *Pinus radiata*, a value of $r = 0.80$ has been measured for lumber cut from mature forests (Leicester 1979a), while a value of $r = 0.35$ has been obtained for immature forests (Anton 1979). Finally, some estimate must be made of r_e, the measure of error in the operational measurement of the grading parameter G. Only limited information is available on this aspect (Leicester 1979b, Madsen 1980). Once the most suitable grading parameter G has been found, a machine is chosen to match the throughput rate of the mill, and N, the number of evaluation tests to be undertaken, can then be optimized.

Although equation (3) provides a useful starting point for assessing the economic feasibility of a grading scheme, there are several other matters which frequently dominate the choice of grading method. One is the existence of grades that are already being marketed by a competing species. This will produce a bias in the market value of specific grades. Another aspect is the severe constraints on choice imposed by limitations in the availability of technology and finance.

Finally, a factor that is sometimes not anticipated is the effect of start-up time. For example, a typical small timber laboratory with an operating personnel of three will complete evaluation tests on some 10,000 sticks of structural lumber per year. With this restriction on the evaluation rate, any grading procedure that is tied to extensive structural lumber testing will not be feasible for mills that process several hundred species.

6. THE LONG TERM FUTURE

As mentioned earlier, the prognostication for the immediate future is for an increase in the quantity of timber to be graded and for a greater variety of situations in which grading will be undertaken. There will probably also be a more systematic approach to the assessment and development of grading methods through the use of models such as the one sketched in this paper. In the longer term future it is probable that the grade verification procedure will be highly codified and rationalized to ensure that the design strength characteristics of timber have the same reliability as those of other structural materials. Furthermore, because of their importance, the various grade verification procedures are likely to be coordinated internationally through the International Standards Organization. There is a possibility that this will be done within a strength grouping format.

Because there will be an increased demand for grading under a wide range of resource and marketing situations, there will also be a wide range of grading systems used. Based on the mathematical model discussed in Section 2.3, the following predictions may be made of the probable characteristics of future grading systems for three interesting situations:

(a) Where few species and large volumes are involved, such as occurs with plantation grown material, the grading systems will be undertaken by electro-mechanical methods characterized by high throughput rates and complex but efficient grading parameters. Extensive evaluation testing will be undertaken, perhaps even on an annual basis, to ensure that the grading parameters used are the best ones possible.

(b) Where multiple-species forests are the source of raw material for mills with easy access to finance and highly developed technology, the grading system will again involve high throughput machines with the ability to operate with complex grading parameters. However, because of the numerous species involved, many of which will be unidentified or available only in small volumes, even a modest amount of evaluation testing for each species may not be economically feasible. Consequently the evaluation of the grading parameters will probably be made through the use of on-line proof testing facilities. These facilities will need to be capable of applying bending and tension loads, and to have rapid response characteristics so that the applied load may be changed from one stick of lumber to the next.

(c) Where multiple species forests are associated with mills that have poor access to finance and technology, the most appropriate grading system will be proof testing based on a simple bending test. Such mills will produce a very limited set of grades, possibly only one for each group of similar species.

7. CONCLUSIONS

In the future, the grading of structural lumber will be associated with a wide range of situations. Because of this, many types of grading systems will be used, and mathematical models of the type described in Section 2.2 will be required to evaluate the cost-effectiveness of various proposed systems. Some prognostications of typical grading systems for the future are given in Section 6.

8. LITERATURE CITED

American Society for Testing and Materials (1980). ASTM D245-74. Standard methods for establishing structural grades and related allowable properties for visually graded lumber. Annual Book. Part 22. USA.

Anon. (1972). New Australian stress grading machine for world markets. Australian Timber Journal, April, pp.65-69.

Anon. (1981). Proof grader a success. Australian Forest Industries Journal, February, p.62.

Anton, A. (1979). The grading of 13-year old radiata pine. Proceedings of 19th Forest Products Research Conference, Melbourne, November.

British Standards Institution (1973). British Standard 4978. Specification for timber grades for structural use. British Standards Institution, London.

Bryant, P.A.V. (1978). The stress grading of S.A. pine. HOUTIM. CSIR National Timber Research Institute, Pretoria, June.

Curry, W.T., and Tory, J.R. (1976). The relation between the modulus of rupture (ultimate bending stress) and modulus of elasticity of timber. Current Paper 30/76. Building Research Establishment. Princes Risborough, UK, April.

Dawe, P.S. (1964). The effect of knot size on the tensile strength of European redwood. Wood, November, pp.49-51.

Ellingwood, B., Galambos, T.V., MacGregor, J.G., and Cornell, C.A. (1980). Development of a probability based load criterion for American National Standard A58. NBS Publication 577. US Dept of Commerce, Washington, June.

Espiloy, Jr. E.B. (1978). Strength grouping of Philippine timbers for utilisation of lesser-known species. Forest Products Research and Industrial Development Commission, National Science Development Board, Technical Note No. 187, March.

Gerhards, C.C., and Ethington, R.L. (1974). Evaluation of models for predicting tensile strength of 2- by 4-inch lumber. Forest Products Journal, Vol.24, No.12, December, pp.46-54.

Ince, P.J. (1979). Cost of grading lumber by the machine-stress-rating process. Forest Products Journal, Vol.29, No.10, October, pp.80-83.

Innotec Oy (1980). Stress grading through direct, non-contacting strength measurement with the Finnograder. Innotec Oy. Luoteisrinne, Finland.

Leicester, R.H. (1979a). In-grade structural properties of some Australian timbers. 19th Forest Products Research Conference, Melbourne, November.

Leicester, R.H. (1979b). Proof grading, a practical application of reliability theory. Proceedings of Third International Conference on the Applications of Statistics and Probability in Soil and Structural Engineering, Sydney, January-February, pp.263-277.

Leicester, R.H. (1980). Grouping and selection of species for structural utilization. Eleventh Commonwealth Forestry Conference, Trinidad, September.

Madsen, B. (1978). In-grade testing, problem analysis. Forest Products Journal, Vol.28, No.4, April, pp.42-50.

Madsen, B. (1980). Parameters affecting the efficiency of mechanical grading. Proceedings of IUFRO Conference, Oxford, UK, April, pp.175-192.

Madsen, B., and Knuffel, W. (1980). Investigation of strength-stiffness relationship for South African timbers as it relates to mechanical grading. Proceedings of IUFRO Conference, Oxford, UK, April, pp.125-173.

Orosz, I. (1969). Modulus of elasticity and bending strength ratio as indicators of tensile strength of lumber. Journal of Materials, Vol.4, No.4, pp.842-864.

Pong Sono (1974). Merchantable timbers of Thailand. Symposium on Research and Marketing of South East-Asian Timbers and Timber Products, organized by the German Foundation for International Development (Berlin) and the Government of the Republic of the Philippines, Manila and Los Banos, Philippines, 19-23 November, pp.120-165.

Schniewind, A.P., and Lyon, D.E. (1971). Tensile strength of redwood dimension lumber. II. Prediction of strength values. Forest Products Journal, Vol.21, No.8, August, pp.45-55.

Sexsmith, R.G., and Fox, S.P. (1978). Limit states design concepts for timber engineering. Forest Products Journal, Vol.28, No.5, May, pp.49-54.

Standards Association of Australia (1975). Australian Standard AS 1720-1975. SAA Timber Engineering Code. North Sydney.

Standards Association of Australia (1979a). Australian Standard AS 1684-1979. SAA Timber Framing Code. North Sydney.

Standards Association of Australia (1979b). SAA Miscellaneous Publication MP 45-1979. Report on strength grouping of timbers. North Sydney.

Standards Association of Australia (1979c). Australian Standard 1490-1973. Visually stress graded radiata pine for structural purposes. North Sydney.

Walters, E.O., and Westbrook, R.F. (1970). Vibration machine grading of southern pine dimension lumber. Forest Products Journal, Vol.20, No.5, May, pp.24-32.

Wong, W.C., and Wong, C.N. (1980). Grouping species in plywood manufacture. Proceedings of Eleventh Commonwealth Forestry Conference, Trinidad, September.

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APPENDIX A

AUSTRALIAN STANDARD

RULES FOR EVALUATION OF GRADED TIMBER

AS -1983

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1. SCOPE AND METHOD

This Part describes methods for evaluating the structural properties of graded timber. In turn, these methods may be used as a basis for describing the structural properties that are to be expected of structural timber.

It should be emphasised that it is not necessary to use the methods to be described to verify the correctness of the specified structural properties of all structural timber. Rather the method should be used primarily to assess specific grading techniques and, in unusual cases, to resolve any doubts concerning the specified design properties of particular populations.

Specifically, this Part is concerned with the evaluation of the basic working stresses, the design modulus of elasticity and the stress grade of a given reference population of timber. This evaluation is based notionally on a set of standard tests. These tests are described in terms of standard sampling procedures and testing configurations. Methods of using data obtained from non-standard evaluation procedures are discussed in Appendix D.

In the following, the basic working stress or design modulus of elasticity will be derived in the following form,

$$R^* = R_k/\gamma \qquad (1.1)$$

where

R = strength or modulus of elasticity

R_k = characteristics value of R

R^* = basic working stress or design modulus of elasticity

γ = load factor

The characteristic value R_k will be taken to be the five-percentile value measured on a large sample taken from a reference population. Thus the design value R^* applies specifically to this reference population.

Note. It should be apparent from the definition of the characteristic values R_k, that the design values R^* apply to populations of timber and not to single sticks.

The standard test configurations to be specified for the measurement of R_k have been chosen to simulate in-service conditions. Thus typical sizes, spans and loading arrangements are recommended, and the test specimens are cut from locations selected at random from within the sticks of timber samples.

2. DEFINITIONS AND NOTATION

2.1 Definitions

Reference Population. This is the population of structural timber for which the design properties are being evaluated. The reference population is defined in terms of the source of timber, the moisture content and size of sticks, and the method of grading used.

Standard Sample. A sample of size greater than 400 sticks and having structural properties that are statistically similar to those of the reference population.

Standard Test Configuration. Test configuration specified for the standard evaluation procedure.

2.2 Notation

D = depth of member
$F_R(x)$ = cumulative distribution function of R at the value x
l = standard span
l_D = non-standard span
L = standard graded length of sticks
L_0 = non-standard graded length of sticks
N = sample size
R = strength or modulus of elasticity
R_k = characteristic value of R
R^* = basic working stress or design modulus of elasticity
$R_{0.05}$ = five-percentile value of R

V_R = coefficient of variation of R

α = bias factor for location of defects, Appendix B.4

γ = load factor

3. REFERENCE POPULATION

The reference population must be clearly defined since it is only to this population that the derived design properties apply. It is the population that is supplied commercially as a specific grade of timber obtained from a specific source. A reference population may be graded from a single species or a defined mixture of species. Typically the timber is limited to the trees cut over a period of 5 to 10 years from forests of a specified geographical region. Other parameters that must be specified in the definition of the timber of a reference population are moisture content, size (both cross-section and length), grade and the method of grading.

Note. From the above it follows that if the reference population changes, then the structural properties may also change and hence should be reassessed.

4. SAMPLING

The sample to be tested must be chosen so as to be representative of the reference population. Considerable care must be taken to ensure that all obvious structural characteristics are distributed within the sample in much the same frequency as they are to be found within the reference population.

Note. Methods of processing data obtained from small samples are described in Appendix B.

In order to simulate in-service conditions, test specimens must be cut from random locations within the sticks of the timber sample. Details of the method for doing this are given in Appendix C.

5. STANDARD TEST CONDITIONS

The standard test configurations for evaluating basic working stresses and the modulus of elasticity are shown in Figures 5.1 to 5.4. For the tests to evaluate bending and shear strengths, the choice of the edge to be stressed in tension should be selected at random. The specimens shall be conditioned to a temperature of $20^{\circ} \pm 2^{\circ}C$ and shall be loaded at a rate that will cause failure in 3 to 5 minutes.

Note. The selection of a random edge for bending tests is intended to simulate in-service conditions.

Note. The configuration required for the compression test shown in Figure 5.3 is awkward to achieve because of the requirement of lateral restraints. However, this test may be simulated with sufficient accuracy by testing a short specimen selected from the original test specimen in such a way as to include the worst visual defect at its mid-length. A value of eight would be suitable for the length to thickness ratio used in this test.

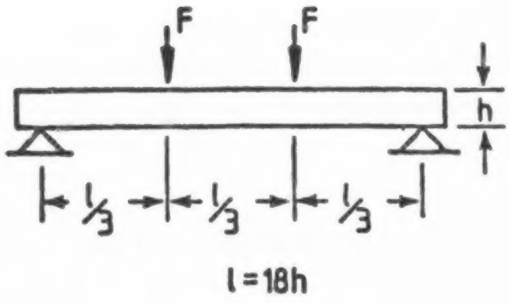

Fig. 5.1 Standard test configuration for measurement of bending strength and modulus of elasticity

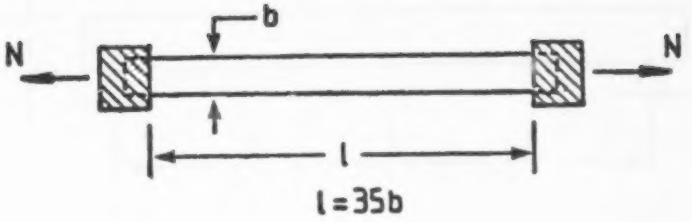

Fig. 5.2 Standard test configuration for measurement of tension strength

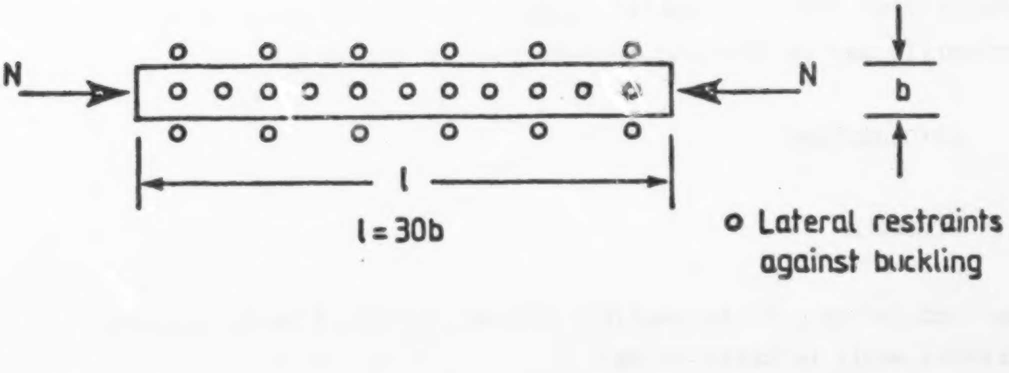
N
N
b
l
l=30b
o Lateral restraints against buckling

Fig. 5.3 Standard test configuration for measurement of compression strength

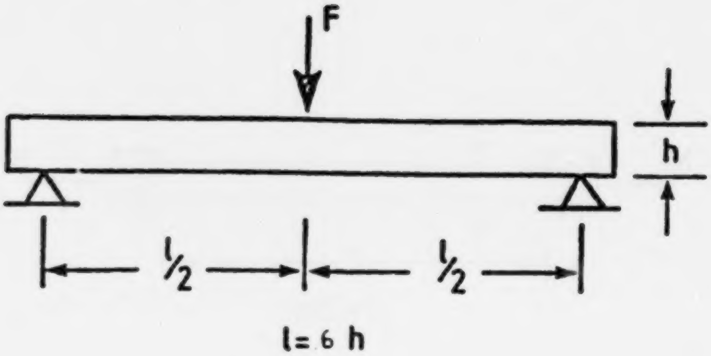

Fig. 5.4 Standard test configuration for measurement of shear strength

6. CHARCTERISTIC VALUES

Characteristic values will be defined as the five-percentile values measured in standard tests.

Note. Five-percentile values can be measured directly from samples greater than 1000. For smaller samples, a safe estimate of the five-percentile may be obtained as described in Appendix B.

7. LOAD FACTORS

7.1 Strength

The load factor γ to be used for the derivation of basic working stresses shall be taken to be

$$\gamma = 1.75\ (1.3 + 0.7\ V_R) \qquad (7.1)$$

where V_R is the coefficient of variation of the strength R measured in the tests.

Note. The factor 1.75 is the value of modifier K_1 given in Table 2.5 of Part 1 of this code, to define the relationship between the three minute design strength and the basic working stress.

The true factor of safety is $1.3 + 0.7 V_R$. This value has been chosen to match existing practice.

If the timber is of a species, moisture content, grade and size such as to be prone to checking in service, the load factor defined by equation (7.1) shall be increased by a factor of 1.5 in deriving the basic working stress in shear.

7.2 Stiffness

For evaluating the design modulus of elasticity, the load factor to be used is

$$\gamma = 0.7 \tag{7.2}$$

Note. Equation (7.2) has been chosen to lead to a design modulus of elasticity that is roughly equal to the mean value for the case of a single grade of a single species. For the case of mixtures of species, the intent is to ensure that for no sticks is the modulus of elasticity excessively below the design value.

8. DESIGN VALUES

As noted earlier in Section 1, the basic working stress and design modulus of elasticity, denoted by R*, are to be derived from

$$R^* = R_k/\gamma \tag{8.1}$$

where R_k is the characteristic value obtained as described in Section 6 or Appendix A, and γ is the load factor specified in Section 7.

9. STRESS GRADE

For any particular stress grade, the associated values of basic working stress and modulus of elasticity are specified in Table 2.3 of Part 1 of this code. For large size members these values must be modified according to the factor K_{11} given in Table 2.10 of Part 1.

In order to classify the reference population into a stress grade, preliminary classifications are first made for each of the individual properties. On the basis of these preliminary classifications, the final classification for the reference population is made according to the rules shown in Table 9.1.

TABLE 9.1
RULES FOR STRESS GRADE CLASSIFICATION OF THE REFERENCE POPULATION

Preliminary classification				Resultant stress grade for reference population
Bending strength	Tension strength	Compression strength	Modulus of elasticity	
F	F	F	F	F
F	F+1	F-1	F-1	F
F	F-1	F	F+2	F

Note: The stress grade F+1 is one grade higher than the stress grade F

10. ASSESSMENT REPORTS

Reports of assessments of structural properties shall include the following information:

(a) Definitions of the reference population. (These will include the source of timber, method of sampling, moisture content, stick dimensions, stick length, and method of grading).
(b) Test configurations.
(c) Data obtained.
(d) Estimates of means, coefficients of variation and five-percentile values.
(e) Derived values of basic working stresses and/or the design modulus of elasticity.

APPENDIX B

MODIFICATION FACTORS FOR NON-STANDARD CONDITIONS

B.1 GENERAL

When non-standard conditions are used for the structural evaluation of characteristic values, it is necessary to make a translation of the data to estimate the characteristic value that would be obtained under standard conditions. The following describes modification factors that may be applied when the method of evaluation is close to that recommended as standard.

B.2 SAMPLE

It is recommended that the sample size used shall be not less than 50 for the evaluation of basic working stresses, and not less than 20 for the evaluation of the design modulus of elasticity.

Note. With small sample sizes it is particularly important to ensure that they are representative of the reference population.

If the sample size is at least 50, then the characteristic value is given by

$$R_k = R_{0.05} [1 - 3 V_R/\sqrt{N}] \qquad (B.1)$$

where

$R_{0.05}$ = the five-percentile mesured directly from the data
V_R = coefficient of variation of R
N = sample size

Note. Equation (B.1) leads to an estimate, with 80 per cent confidence, of the true characteristic value.

When a sample size of less than 50 is used for the modulus of elasticity, and when the reference population is a single grade of a single species, then the following may be used as an alternative method of estimating the characteristic value of the modulus of elasticity,

$$R_k = 0.6 \bar{R} \qquad (B.2)$$

where R_k is the mean value of R.

Note. Equation (B.2) cannot be used with mixtures of species, because for the case of large coefficients of variations, it would lead to the possibility of sticks having a modulus of elasticity far below the design value.

B.3 TEST SPECIMEN LENGTH

B.3.1 Effect on Basic Working Stresses

If the length of the test specimen is shorter than the standard length, then the measured strength values will be increased. For this case, a conservative correction will be obtained if the characteristic value is not taken to be the five-percentile value, but instead is defined according to the following equation

$$F_R(R_k) = 0.05\ (l_o/l) \qquad (B.3)$$

where

$F_R(x)$ = cumulative distribution function of the test values of R at the value of x

l = span in standard test

l_o = span in actual test.

An example of the application of equation (B.3) is given in Appendix D.

Note. The modification given by equation (B.3) is derived on the assumption that the strengths of segments of timber cut from the same stick are perfectly uncorrelated.

B.3.2 Effect on Modulus of Elasticity

The modification to the modulus of elasticity measured with non-standard spans shall be given by

$$R = (R_o/1.04)\ [1 + 14\ (D/l_o)^2] \qquad (B.4)$$

where

R = the derived value for a standard span

R_o = the value measured on the non-standard span

D = member depth

l_o = non-standard span.

Note. The design modulus of elasticity specified in Table 2.3 of Part 1 of this code contains an allowance for the deformation due to shear. This allowance is based on the value for a standard span-to-depth ratio. Equation (B.4) provides a comprensation for the fact that shear deformations form a greater proportion of the total deflection as the span-to-depth ratio is reduced.

B.4 GRADED LENGTH

If the graded length of timber is longer than the standard value, then the measured strength values will be increased. For this case, a conservative correction will be obtained if the characteristic value of strength is not taken to be the five-percentile value, but instead is defined according to the following equation

$$F_R(R_k) = 0.05\ (L/L_o) \tag{B.5}$$

where

$F_R(x)$ = cumulative distribution function of the test values of R at the value x

L = the standard graded lengths of timber

L_o = the graded lengths used for the actual test.

Note. If both the graded lengths of timber are too long and the test spans are too short, then from equations (B.3) and (B.5) the appropriate correction is obtained by choosing R_k so that $F_R(R_k) = 0.05(l_o/l)(L/L_o)$.

Note. There is no effect of graded length on the modulus of elasticity.

B.5 DEFECT LOCATION

In a standard test, the locations of defects are assumed to occur at random locations. Frequently the test specimen is deliberately chosen so that it contains the worst defect in the stick; furthermore, this defect is then located on the tension edge during bending strength tests. For this case the corrected characteristic value R_k is obtained from

$$R_k = \alpha R_{0.05} \tag{B.6}$$

where

$R_{0.05}$ = the five-percentile value measured in the test

α = a factor to account for the bias in the defect location.

In the absence of reliable information, the conservative values of α shown in Table B.1 may be used for evaluating the basic working stress in bending of softwood timber species.

TABLE B.1
VALUES OF PARAMETER α FOR SOFTWOODS

Grade designation	Nominal grade strength (% clear wood strength	α
Str. No. 1	75	1.00
Str. No. 2	60	1.05
Str. No. 3	48	1.10
Str. No. 4	38	1.15
Str. No. 5	31	1.20

B.6 PERCENTILE VALUES

Where characteristic values other than the five-percentile are available, the five-percentile values may be deduced by assuming any reasonable distribution (such as the lognormal distribution) for the structural parameter R.

APPENDIX C

METHOD FOR RANDOM SELECTION OF SPECIMEN LOCATION

In order to choose a random location for a specimen of length l_s in a stick of length L, the end of the specimen should be located at a length l_e from the end of the stick, where l_e is given by

$$l_e = r(L - l_s) \tag{C.1}$$

in which r is a number selected at random from a uniform distribution in the range 0 to 1. The end from which the length l_e is measured is to be chosen at random, possibly by the toss of a coin. The notation is illustrated in Figure C.1. A set of values for the random number r is given in Table C.1.

Frequently, because of the random method of cutting sticks, the complexity involved in using the above procedure for selecting specimen locations may not be warranted. In this case, adequate accuracy is obtained if the specimens are cut either from the centre or from a randomly chosen end.

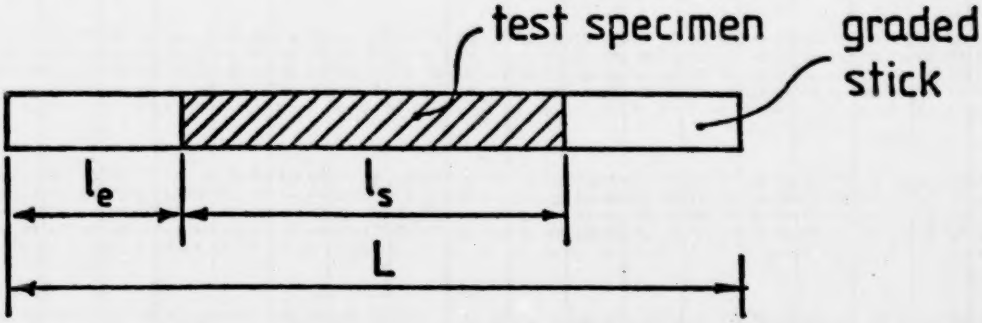

Figure C.1 Notation for specimen selection procedure

TABLE C.1

SET OF RANDOM NUMBERS FROM 0 TO 1

.0389	.2690	.2598	.7565	.2218	.4784	.6209	.0627	.5495	.7975
.0265	.5686	.1492	.6969	.4510	.7378	.5084	.0405	.2665	.2485
.0187	.1914	.2657	.6246	.4916	.9398	.5076	.4058	.9723	.3106
.7573	.3782	.2993	.5359	.6193	.8442	.3420	.2952	.4477	.4987
.6880	.1928	.2247	.5292	.9539	.7492	.9971	.6926	.2175	.2734
.9609	.9481	.9024	.1045	.4947	.2568	.5239	.0998	.4919	.3473
.9809	.0168	.2251	.6057	.6776	.7096	.2347	.6002	.9944	.7502
.6820	.3727	.0095	.4672	.2029	.7247	.5879	.6025	.2132	.4954
.8838	.0321	.1853	.1265	.0096	.1800	.6869	.3536	.3719	.4293
.6707	.1536	.3884	.3668	.9685	.3609	.6259	.2556	.4963	.0185
.6413	.0240	.6885	.8155	.9753	.9666	.1368	.9563	.0335	.2696
.0276	.3019	.3668	.2277	.8708	.7778	.7034	.4652	.6086	.4876
.7415	.0535	.7481	.7402	.1206	.1885	.5719	.7946	.2890	.1573
.3532	.4586	.4838	.6949	.1146	.1307	.2427	.4772	.7972	.2281
.4729	.2975	.4141	.4479	.9765	.2716	.1801	.6429	.5559	.9792
.8602	.3527	.6381	.4791	.0239	.7929	.2125	.2826	.9001	.3244
.3992	.2132	.5129	.5839	.9685	.1409	.5520	.3059	.6264	.3116
.7742	.8936	.0485	.7960	.7738	.5109	.8762	.7865	.1821	.2495
.7374	.3239	.3757	.2519	.5861	.8253	.8134	.6661	.2018	.5005
.0523	.4225	.6865	.6614	.0198	.0035	.5592	.8996	.3360	.9759
.0754	.1930	.5918	.5764	.3890	.0487	.9581	.3243	.0007	.6455
.7960	.6554	.6704	.3738	.0376	.0374	.8635	.8817	.7571	.4982
.2871	.3831	.9545	.5564	.8053	.1328	.2259	.6166	.7741	.1552
.4870	.8398	.6655	.9804	.7133	.1330	.9003	.6908	.9322	.9450
.4906	.3389	.2870	.8944	.9516	.0430	.4303	.5040	.7522	.9952
.0210	.5122	.2521	.3867	.5933	.8841	.7758	.9322	.1516	.6603
.4895	.2945	.7912	.5339	.6074	.2186	.4368	.3141	.8228	.4804
.0792	.5745	.6412	.6695	.0324	.1258	.5351	.1309	.7519	.2357
.8040	.7847	.3883	.5326	.1557	.0862	.8906	.9005	.9413	.6474
.2365	.4603	.5080	.5418	.0231	.9400	.2709	.5174	.6821	.6285
.0537	.6362	.1447	.4327	.3686	.7760	.7261	.4864	.1854	.2462
.8512	.9053	.1226	.6837	.2408	.7006	.7871	.5554	.2182	.0383
.4418	.0869	.4819	.8850	.3929	.6716	.3082	.9916	.2239	.7583
.6674	.9102	.7359	.7583	.8220	.5848	.1897	.4841	.9662	.6468
.1983	.8082	.4122	.0273	.6602	.3117	.4317	.3923	.8872	.3262
.5540	.2214	.2018	.5453	.9832	.5027	.5685	.7995	.6257	.8729
.2874	.1295	.4777	.8894	.6110	.3257	.9784	.7215	.7192	.9266
.2324	.0691	.3741	.1273	.2771	.1559	.5308	.6728	.0482	.3412
.1349	.9253	.2775	.3719	.4354	.3965	.8529	.4414	.6855	.2514
.4534	.7170	.6068	.9893	.5224	.6652	.1398	.7728	.5216	.8924

APPENDIX D
EXAMPLE OF NON-STANDARD TEST

D.1 DATA

The following is an example of the derivation of a basic working stress in bending.

The measured bending strength R of a particular softwood timber of structural grade No. 4 is given in Table D.1. The derived coefficient of variation V_R is 0.37.The cross-section is 100 x 35 mm, and the test span used was 1500 mm. The test specimens were selected and biased so that the worst visual defect of each stick was location in the maximum tension zone of a test specimen.

D.2 DERIVATION

The cumulative distribution function of R is computed as shown in Table D.1 and graphed in Figure D.1. Because the span for a standard test as shown in Figure 5.1 is 1800 mm, the first estimate of the characteristic value for a standard span is obtained from equation (B.3) which leads to

$$\begin{aligned} F_R(R_k) &= 0.05\ (1500/1800) \\ &= 0.042 \end{aligned}$$

From the graph in Figure D.1, this leads to

$$R_k = 27.0 \text{ MPa}$$

In fact R_k needs to be modified further. One modification is the location bias factor $\alpha = 1.15$ given in Table B.1.

The modification factor given in equation B.1 for small sample size is

$$1.0 - (3.0)(0.37)/\sqrt{109} = 0.89$$

Thus the modified estimate of the characteristics value is

$$R_k = 27.0 \times 1.15 \times 0.89$$
$$= 27.6 \text{ MPa}$$

Hence from the definition of the load factor γ in equation (7.1), the basic working stress R* is given by

$$R^* = (27.6/1.75)/[1.3 + (0.7)(0.37)]$$
$$= 10.1 \text{ MPa}$$

A comparison with the basic working stress in bending, given in Table 2.3 of Part 1 of this code, indicates that a preliminary stress grade classification of F8 would be applicable to the bending strength of this reference population.

TABLE C.1
BENDING STRENGTH DATA

Strength ranking i	Bending strength x (MPa)	Cumulative distribution function $F_n(x)$
1	15.7	0.0046
2	19.5	0.0138
3	24.0	0.0229
4	24.2	0.0321
5	27.1	0.0413
6	28.3	0.0505
7	28.6	0.0596
8	29.4	0.0688
9	30.4	0.0780
10	30.6	0.0872
11	31.8	0.0963
12	32.3	0.1055
108	126.9	0.9862
109	138.3	0.9954

N = 109
$F_n(x_i) = [i - 0.5]/N$

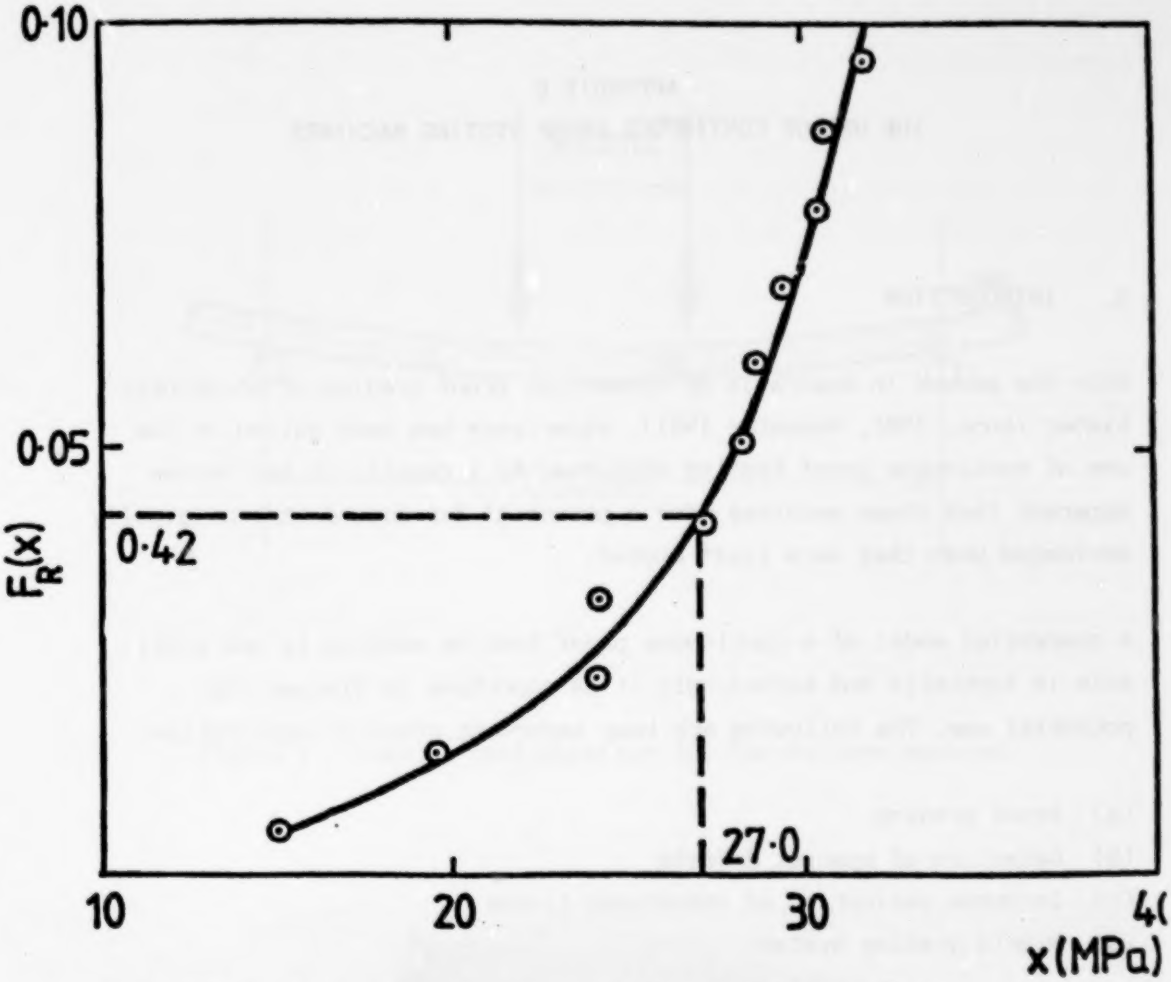

Figure D.1 Cumulative distribution function of measured bending strength

APPENDIX E

THE USE OF CONTINUOUS PROOF TESTING MACHINES

1. INTRODUCTION

With the advent in Australia of commercial proof grading of structural timber (Anon. 1981, Woooster 1981), experience has been gained in the use of continuous proof testing machines. As a result, it has become apparent that these machines have a potential far beyond that originally envisaged when they were first mooted.

A commercial model of a continuous proof testing machine is now available in Australia and accordingly it is opportune to discuss its potential use. The following are four important areas of application.

(a) Proof grading
(b) Detection of special defects
(c) In-grade evaluation of structural timber
(d) Hybrid grading systems.

2. THE COMMERCIAL MACHINE

The Australian commercial machine (originally designed by TRADAC, Queensland) is constructed by Hilleng Pty Ltd of Brisbane. It applies a continuous bending moment to the timber as it passes through the machine (Plate 1). The loading configuration is shown schematically in Figure 1. The timber is loaded on edge, i.e. it is stressed about its major axis. The machine comes with two interchangeble rams which have load capacities of 20 and 36 kN. The rams are activated by an 800 kPa air pressure source. The feed rate through the machine is about 15 metres per minute. A limit switch removes the load if the timber breaks or deflects excessively.

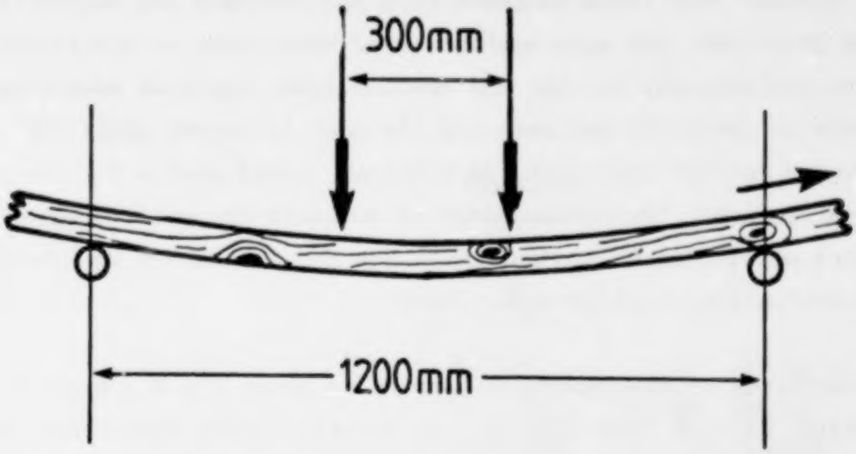
300mm
1200mm

Figure 1 Loading configuration for the Hilleng machine

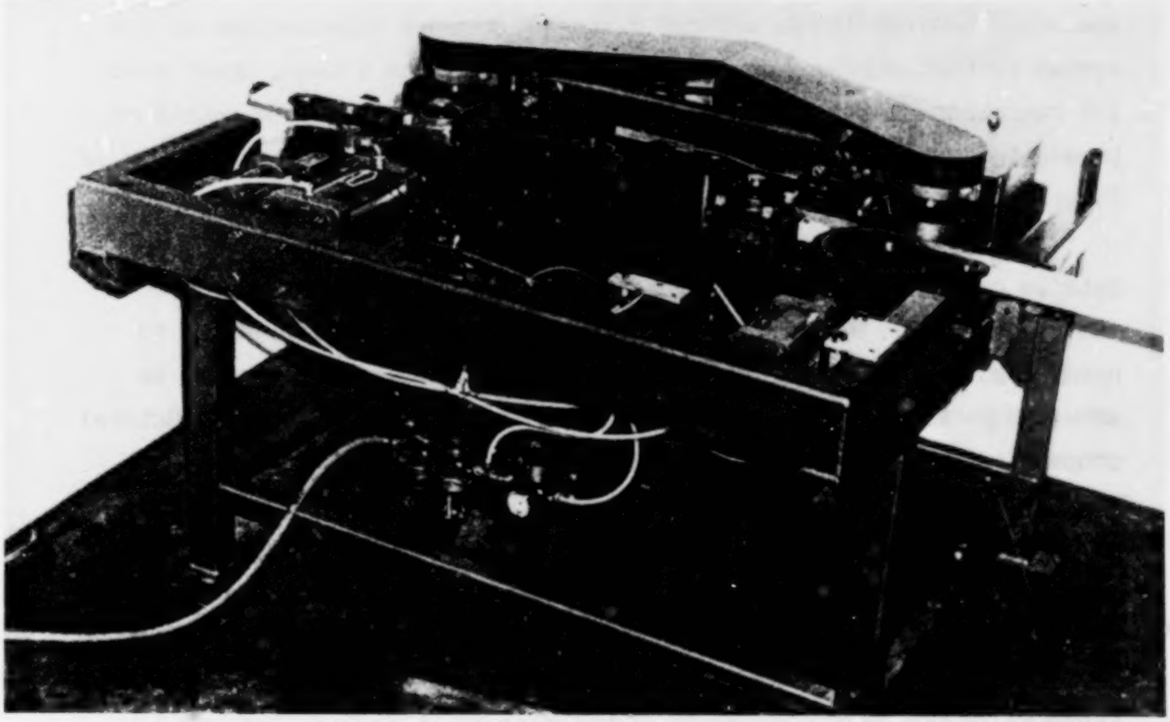

Plate 1 The Hilleng proof grading machine

The machine is sold in two basic models. In the simplest model, and air pressure gauge is used for setting the load. The accuracy of this model under dynamic conditions is about five per cent and the current cost is around $US12,000. The more sophisticated model uses an electronic load cell to continuously monitor the applied load. The load measurement accuracy is about one per cent and the cost is around $US14,000. A read-out device for the load cell, if required, would cost a further $2,000. In general terms, the simple model is suitable for proof grading while the more sophisticated model is suitable for laboratory work such as in-grade evaluation of structural timber.

The dimensions of the assembled machine is about 2 m x 2 m x 1.5 m, and the weight is less than 1000 kg. It is sufficiently small that it can be placed on a truck and transported anywhere in Australia; and in fact the prototype model has been used for demonstrations at locations more than a thousand kilometres from its base at Brisbane.

3. PROOF GRADING

The essence of proof grading is to accept a stick of lumber as being of a specified stress grade if it demonstrates an ability to sustain a specified bending stress without distress. A major disadvantage of the system is that only one stress grade is output from a basic proof grading operation. If more than one grade is required, then some method of presorting the timber must be undertaken as illustrated schematically in Figure 2.

Details of the proof grading procedure will be given elsewhere (Leicester 1983). Matters to be considered include the damage due to proof testing, errors by grader operators in choosing the edge to be stressed in tension, and the effect of proof testing on other structural properties such as tension strength and stiffness.

The presorting of timber into grades is undertaken in a variety of ways, depending on the operational procedures used in each particular mill. A visual assessment is usually part of the sorting procedure and can prove highly effective, particularly as the proof testing machine provides a feedback on the on the accuracy of the assessment method. The use of stiffness, averaged along the length of a stick, has yet to be tried, but in theory this can be a method for obtaining timber tailored to meet a specified set of strength and stiffness criteria.

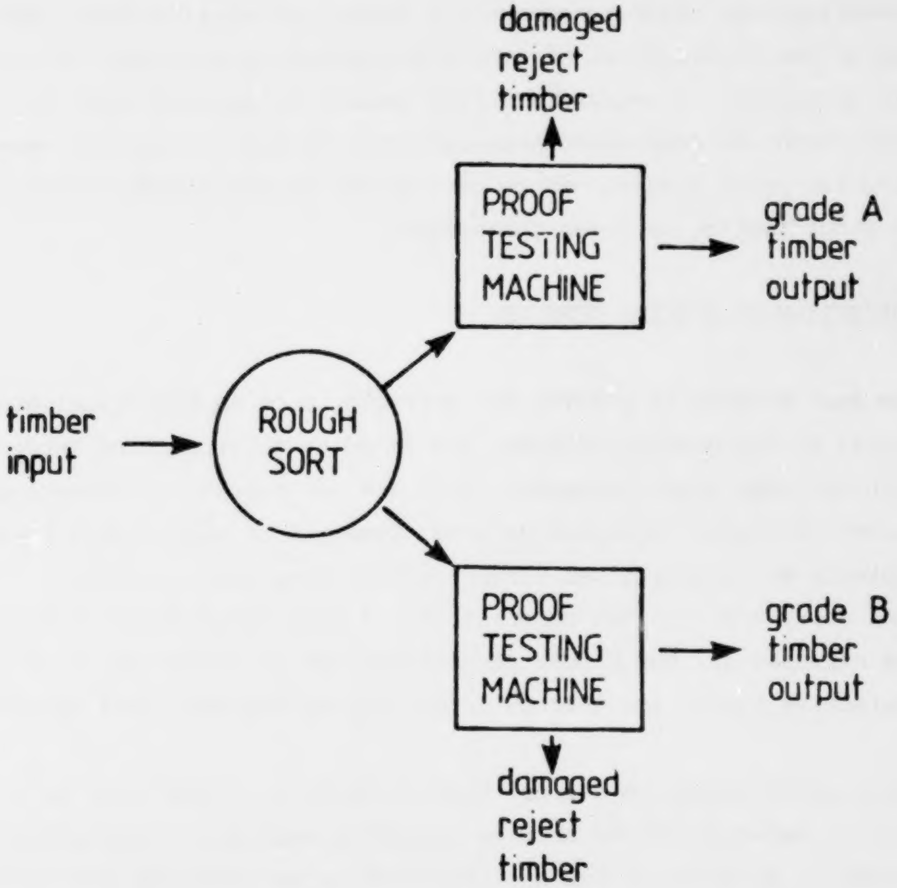
damaged
reject
timber
PROOF
TESTING
MACHINE
grade A
timber
output
timber
input
ROUGH
SORT
PROOF
TESTING
MACHINE
grade B
timber
output
damaged
reject
timber

Figure 2 Proof grading operations to produce two stress grades

Compared with visual grading, proof grading has a considerably greater reliability. It avoids the difficulties invisual grading associated with slope of grain and species identification. The problems associated with species identification are particularly acute where hybrids are involved, or where mills are supplied with numerous species. An example of this occurs in northern Queensland where typically the mills are supplied with more than 100 species.

In comparison with mechanical stress grading, the advantages of proof grading are a lower initial cost, a low technology and a shorter lead time to the commencement of a commercial application. This is because the use of a correlation between strength and stiffness in a mechanical stress grading system means that the method requires extensive laboratory investigations prior to commencing commercial applications; and because of the difficulties of accurately measuring stiffness, a high quality of control is necessary during commercial applications. By contrast, both the assessment procedure and the quality control measures required for proof grading can be carried out fairly rapidly through use of the proof testing machines themselves.

4. DETECTION OF SPECIAL DEFECTS

Because most methods of grading for strength, such as the visual and mechanical stress grading methods, are based on a correlation between strength and some other parameter, they are not capable of detecting many kinds of highly localised defects. Examples of such highly localised defects are poorly glued finger-joints, compression shakes, internal decay and internal borer attack. A continuous proof testing machine stresses all the timber passing through it and hence it will fail defective timber and thereby detect any excessively weak material.

In this type of operation a proof testing machine is used only as a detector of defects, rather than as a grading machine. Consequently a high order of accuracy in the applied loads is not required for this purpose and so a much simpler machine than the one produced by Hilleng will be adequate. Such a machine can usually be fabricated from parts costing only a few hundred dollars.

It is of interest to note that the first continuous proof testing machine in Australia (Straker 1977) and possibly also the first one in the United States (Bolger and Rasmussen 1962) were developed to detect faulty finger-joints.

5. IN-GRADE EVALUATION OF STRUCTURAL TIMBER

It is now generally accepted that it is unreliable to base design recommendations on the properties of small clear timber specimens unless penalties in the form of large safety factors are used to compensate for the uncertainties involves (Leicester and Hawkins 1981, Madsen 1978). Ideally design strength properties should be based on the five percentile strength of graded structural size timber.

Unfortunately in-grade measurements for this require the testing of large samples of timber. Typically a sample of some 10,000 sticks may be required to fully evaluate a single species or species mixture. Under normal circumstances this would take about a year to do in a typical forest products laboratory. Obviously then, this form of timber evaluation is not feasible for areas, such as northern Queensland, where hundreds of species are utilized.

However, in-grade testing for these situations can be made quite feasible through the use of continuous proof-testing machines such as the Hilleng machine described earlier. For this purpose, the proof load is set so as to fail one to five per cent of the material passing through. The actual failure load for any particular stick will be indicated by the read-out from the load cell. Even with the relatively slow Hilleng machine, it is possible to test some 1000-2000 sticks a day in this way.

Even where there is only one major species involved, it is desirable to have access to a continuous proof testing machine. This is because the strength properties of the single species will change from time to time due to changes in the age of the timber cut, changes in aspects of silvicultural practices such as the pruning or fertilisation programs, and changes in the locations from which timber is cut.

In general terms it may be stated that continuous proof testing machines are invaluable where frequent changes are likely to occur in the structural properties of the timber utilised.

Regions or countries that utilise imported timber are particularly prone to rapid changes due to changes in marketing factors beyond their control.

Other sources of change noted in Australia include the introduction of timber from regrowth forests caused by forest fires, timber from fire burnt forests, timber from juvenile plantations timber from borer infested forests, and timber cut from a new species. In such instances, the low cost and short lead-time to utilisation associated with a continuous proof tester would make the difference between utilisation and non-utilisation. For this reason alone the Forestry Commissions (or research laboratory) in every region containing production forestry land should own or have access to a laboratory model of a continuous proof testing machine.

6. HYBRID GRADING SYSTEMS

In looking to the future of the use of continuous proof testing machines, one of the most exciting possibilities to speculate on is that of coupling these machines with other grading machines to form a hybrid grading system (Leicester 1981). As mentioned earlier, current major draw backs to the use of conventional grading systems such as visual or mechanical stress grading include the long lead time required to establish the correct grading parameters, the inability of these systems to cope with frequent changes in material characteristics. However, if a proof tester is coupled on-line so as to sample and test the output of a grader as shown schematically in Figure 3, then grading parameters may be devised and checked instantaneously.

Possibly the most sophisticated hybrid system that is technically within the bounds of current technology would be one that is formed by coupling a high speed electronic grader (Innotec Oy 1980), and a high speed mechanical stress grader (Anon. 1981) with on-line bending and tension proof testing machines as shown in Figure 4. This system would have the capability of work with complex grading parameters, of rapidly evaluating their effectiveness.

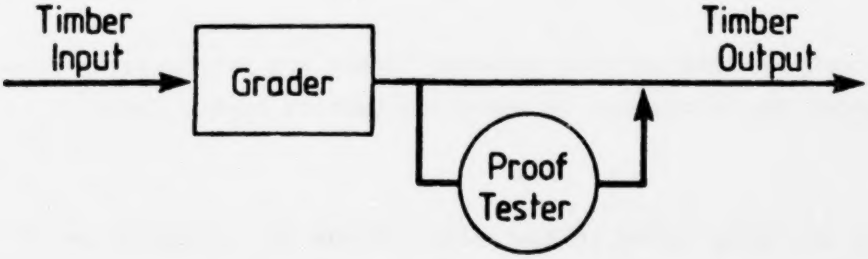

Figure 3 A basic hydrid grading system

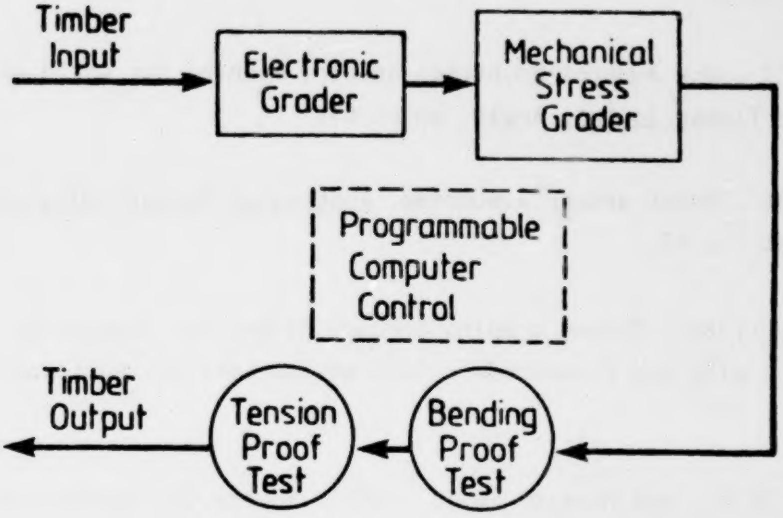
Timber Input
Electronic Grader
Mechanical Stress Grader
Programmable Computer Control
Timber Output
Tension Proof Test
Bending Proof Test

Figure 4 An interactive hydrid grading system

A programmable computer would be used to interpret the grading parameters measured and to make appropriate decisions concerning the grade of timber and the type of proof-testing required. The computer control program need not necessarily be preset, but can be 'learnt' by techniques such as those associated with artifical intelligence systems (Winston 1979).

A hybrid system of this type will represent the ultimate in efficiency and flexibility that is currently possible. It shoulds be able to cope at high-speed with that most difficult of grading problems, the grading of mixtures of unidentified unsorted and untested species.

7. ACKNOWLEDGEMENTS

The author is indebted to A Hill (Hilleng, Queensland), C Mackenzie (TRADAC, Queensland) and W G Poynter (Foxwood, Queensland) for some of the information provided herein.

8. REFERENCES

Anon. (1972). New Australian stress grading machine for world markets. Australian Timber Journal April, pp.65-69.

Anon. (1981). Proof grader a success. Australian Forest Industries Journal Feb., p.62.

Innotec Oy (1980). Stress grading through direct non-contacting strength measurement with the Finnograder. Information leaflet. Luoteisrinne, Finland.

Leicester, R.H., and Hawkins, B.T. (1981). Models for evaluating stress grading. Forest Products Research Conference. Melbourne, Nov.

Leicester, R.H. (1981). The future of grading structural timber. Proceedings of XVII IUFRO World Congress, Kyoto, Japan, Division 5, Sept., pp.35-46.

Leicester, R.H. (1983). Proof Grading. I. An Overview. Division of Building Research Technical Paper (Second Series). CSIRO, Australia. (In press).

Madsen, B. (1978). In-grade testing, problem analysis. Forest Products Journal Vol.28, No.4, April, pp.42-50.

Straker, I. (1977) A simple continuous testing machine. Australian Forest Industries Journal March, pp.39-41.

Winston, P.H. (1979). Artifical intelligence. Addison-Wesley Publishing Co., Sydney.

Wooster, R. (1981). The TRADAC proof grader. Australian Forest Industries Journal April, pp.30-32.

VISUAL GRADING OF TIMBER

J. Hay[1]

INTRODUCTION

It is important with any visual grading process that the method of measuring and assessing defects is consistent. This means that not only should the one grader be able to reproduce almost the same result on regrading, but also that any difference between two graders is marginal. To achieve this requires a form of training to explain the grading terms, the methods of measurement, the relative importance of the various defects and the way in which the results are recorded.

The following notes have been used with considerable success in the training of graders in Victoria, Australia.

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[1] Co-ordinator of Timber Industry Courses, Dandenong College of Technical and Further Education, Victoria, Australia.

TIMBERS IN GENERAL USE

The three main varieties of timber used in the Australian building industry are :

Eucalyptus
Radiata pine, and
Oregon (Douglas fir)

Eucalyptus is the generic or family name for a large number of Australian timber species. As these species usually occur in random mixtures, depending on geographical location, timber produced for the building industry from them is generally not classified into individual species.

Radiata pine and Oregon are ordered and produced according to species.

The following table lists, by species, the various timbers used throughout Australia. Within the table are the strength and stress characteristics determined for each species. Note that these definitions relate to various standards in use throughout Australia and these standards are subject to occasional amendment.

Data obtained from the table must not be considered final. Always refer to the prevailing local standard.

COLOUR CODING

F4	RED
F5	BLACK
F7	BLUE
F8	GREEN
F11	PURPLE
F14	ORANGE
F17	YELLOW
F22	WHITE

Relationship between Visual Structural Grades and Stress Grades

Stress grade is a strength classification for structural timbers, derived by means of either visual or mechanical grading in accordance with the relevant Australian Standards (see Standards Reference List on page 39). The following table shows the stress grades resulting from visual grading to various grade specifications for common structural timbers

SPECIES	AUST STAND	STRENGTH GROUP		VISUAL GRADE					Lyctus Susceptibility (See Note 3)
		Unseas	Seas	Unseasoned		Seasoned			
				Str No 2	Str No 3	Str No 2	Str No 3	Str No 5	
				STRESS GRADE					
Unidentified Aust. Hwds from Vic, Tas, & NSW Highlands	2082	S4	SD4	F11	F 8	F17	F14		S
ASH, ALPINE	2082	S4	SD4	F11	F 8	F17	F14		R
ASH, MOUNTAIN	2082	S4	SD3	F11	F 8	F22	F17		R
ASH, SILVERTOP	2082	S3	SD3	F14	F11	F22	F17		R
BOX, BRUSH	2082	S3	SD3	F14	F11	F22	F17		R
BOX, GREY	2082	S2	SD2	F17	F14	F27	F22		R
GUM, BLUE, SOUTHERN	2082	S3	SD2	F14	F11	F27	F22		S
GUM, GREY, MOUNTAIN	2082	S3	SD2	F14	F11	F27	F22		S
GUM, MANNA	2082	S4	SD4	F11	F 8	F17	F14		S
GUM, RED, RIVER	2082	S5	SD6	F 8	F 7	F11	F 8		S
GUM, SHINING	2082	S4	SD4	F11	F 8	F17	F14		S
GUM, SPOTTED	2082	S2	SD2	F17	F14	F27	F22		S
IRONBARK, RED	2082	S1	SD3	F22	F17	F22	F17		S
JARRAH	2082	S4	SD4	F11	F 8	F17	F14		R
MAHOGANY, SOUTHERN	2082	S2	SD3	F17	F14	F22	F17		R
MESSMATE	2082	S3	SD3	F14	F11	F22	F17		S
PEPPERMINTS, VAR.	2082	S4	SD4	F11	F 8	F17	F14		S
PINE, RADIATA*	1490		SD6				F 8	F 5	R
STRINGYBARK, BROWN	2082	S3	SD3	F14	F11	F22	F17		R
STRINGYBARK, RED	2082	S3	SD4	F14	F11	F17	F14		S
STRINGYBARK, WHITE	2082	S3	SD3	F14	F11	F22	F17		R
STRINGYBARK, YELLOW	2082	S3	SD3	F14	F11	F22	F17		R
TALLOWWOOD	2082	S2	SD2	F17	F14	F27	F22		S
KAPUR	2082	S3	SD3	F14	F11	F22	F17		R
KWILA (MERBAU)	2082	S2	SD2	F17	F14	F27	F22		S
FIR, DOUGLAS (OREGON) +	2440	S5	SD5	F11 Sel. Eng.	F 8 Std. Eng.	F 7 Sel. Mer.	F 5 Merch.		R
HEMLOCK, WESTERN (HEMFIR ETC)	2440	S6	SD6	F 8 Sel. Eng.	F 7 Std. Eng.	F 5 Sel. Mer.	F 4 Merch.		R

* In seasoned condition as required by AS1490. (Grades apply to treated and untreated Radiata Pine).

+ Grading applies to Oregon from the West Coast of North America.

LEGEND:

Visual grades defined in the Australian Standards are as indicated below:

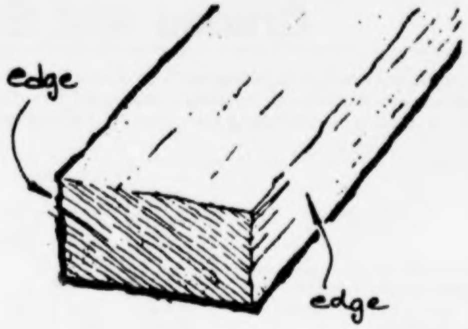
Str. No. 2: Structural Grade No. 2
Sel. Eng.: Select Engineering Grade
Std. Eng: Standard Engineering Grade
Str. No. 3: Structural Grade No. 3
Sel. Merch.: Select Merchantable Grade
Merch.: Merchantable Grade

NOTE:

1. The current standards for visual grading of softwoods use differing systems of grade classification. For simplicity, the structural grades of softwoods have been classified here in the same way as hardwoods.
   The structural grade No. 5 shown is a theoretical grade only and has been included to assist this classification. The grade names for oregon and western hemlock have also been included.
2. Hardwoods are usually marketed as a mixed species and only in special circumstances can particular species be obtained. Supply availability should be determined before design documentation or specification.
3. Lyctus susceptibility R = resistant S = susceptible.

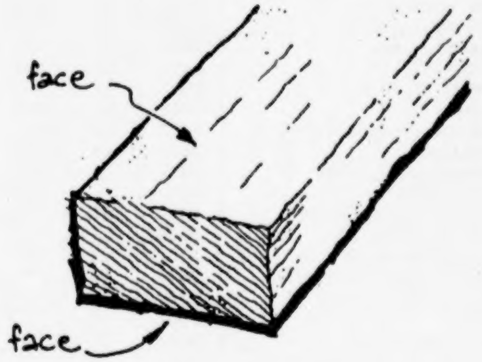
TERMINOLOGY

Each piece of timber has two faces, two edges and two ends. Its cross-sectional area is obtained by multiplying the width of one edge by the width of one face.



- **EDGE**

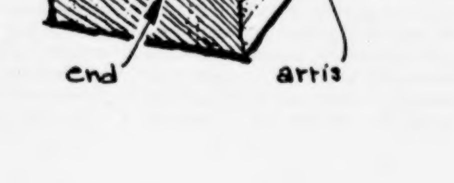
Edges are the two narrow surfaces of one piece of timber. They extend throughout its length.



- **FACE**

The faces are the two wide surfaces of the piece of timber. They extend throughout its length.

A piece of timber may contain sapwood, truewood or heart, or combinations of these.



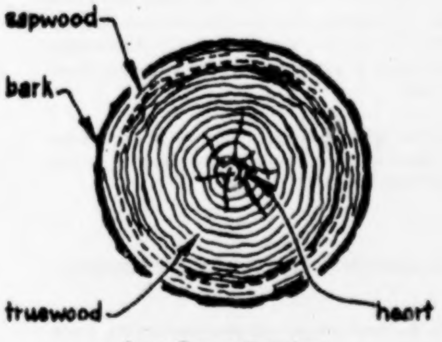
- **ENDS**

The ends are the two remaining surfaces. They extend across the width of the piece of timber.

- **ARRIS**

An arris is the boundary between an edge and a face. Each piece of timber has four arrises.

6.5



Log Cross Section

● SAPWOOD

Sapwood is composed of living wood tissue. It forms the outer layers of a tree trunk and stores and conveys food material within the tree.

● TRUEWOOD

Truewood is wood tissue from which food material has been removed or converted to hard tissue. It constitutes the bulk of the tree trunk and lies under the sapwood.

● HEART

Heart is the material in the centre of the trunk. It is composed of brittle dead wood tissue, and is surrounded by the truewood.

The amount of sapwood, truewood or heart in a piece of timber depends on the size of the original log and the position within the log from which the piece was cut. A piece of timber may be cut from a log by backsawing or quartersawing.

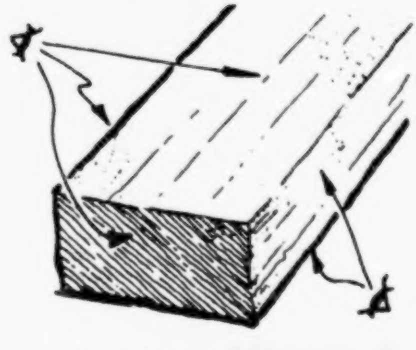
VISUAL STRESS GRADING

Visual stress grading is a technique for grading timber to meet building standards, without the use of mechanical equipment. Success depends on careful visual inspection of each piece of sawn timber, recognition of any defects within that piece and evaluation of their significance with reference to the appropriate standard.

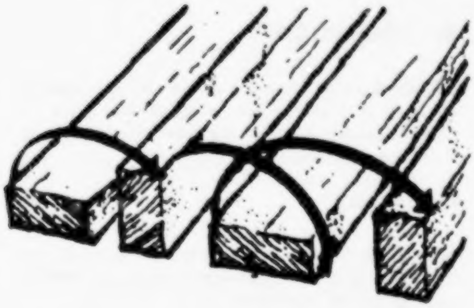
The grading of timber cannot be considered an exact science because it is based on a visual inspection of each piece and thus relies to an extent on the judgement of the grader. These rules provide for a 5% variation between graders. While definite and complete descriptions of timber quality are usually given, it is not feasible or practicable to be fully explicit in every respect, but defects capable of physical measurement shall be judged solely on the basis of their dimensions.

Grade descriptions based upon the poorest pieces allowed in each grade are desirable in the interest of keeping the rules simple and facilitating the work of the grader. As a result the average quality in any grade is far better than the minimum described.

The stress grader needs a straight edge, a steel tape (calibrated in millimetres) and a strong pocket knife.



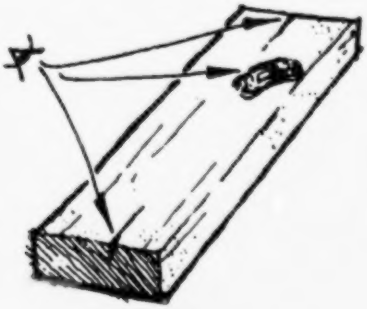
Check Each Surface



Rolling the Piece of Timber

● HANDLING THE PIECE OF TIMBER

Both faces, both edges and each end of the piece of timber must be inspected. This usually involves inspection from one end, rolling of the piece to allow the length of each edge and face to be examined.



Look for the Most Obvious Defect

● INSPECTION

Inspection of a piece of timber should be thorough, but for reasons of economy, it must be performed as rapidly as possible. Roll the piece, check the edge and face widths, then look for the most obvious defect. If this is sufficient to reject the piece, ignore any other defects present.

Rework of many rejected pieces is possible. Note this in accordance with mill practice, and proceed to the next piece without delay.

When in doubt, store the piece in a separate stack for future re-evaluation. Ask your supervisor for assistance during this re-evaluation exercise.

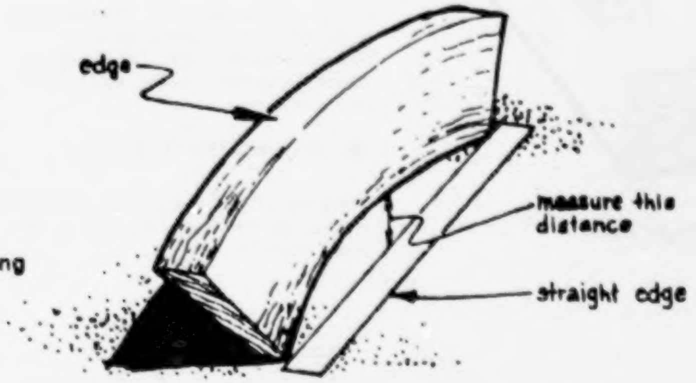
DEFECTS

● SPRING

Spring is curvature along the edge of a piece of timber. To measure, lay the concave edge of the timber against a straight edge, measure the greatest distance between the straight edge and the face with a tape.

When related to the dimensions of the piece, this measurement will indicate acceptability of the **spring**

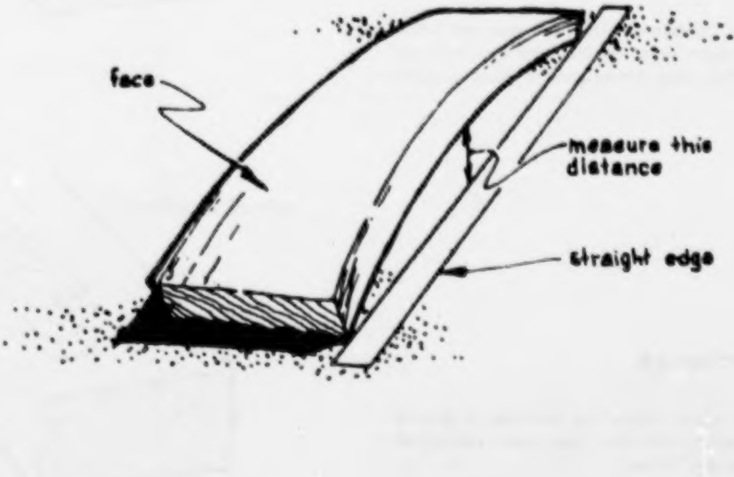
With practice **spring** may be accurately checked, without measurement, by sighting along the concave arris.


edge
measure this distance
straight edge

Spring

● BOW

Bow is curvature along the faces of a piece of timber. It is measured in the same way as spring, and with practice may also be accurately checked, without measurement, by sighting along the concave arris.

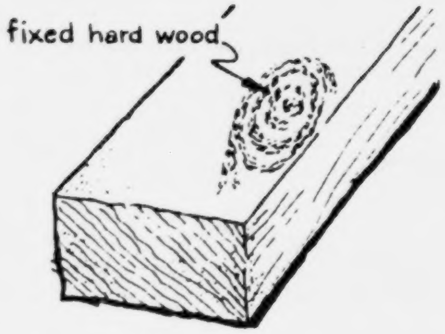

face
measure this distance
straight edge

Bow

KNOTS

Knots are inclusions of branch growth sections in timber pieces. There are several types of **knot**, and the method of evaluation of each type differs. It is important that the timber grader is able to recognise each type. The acceptability of each type of **knot** is calculated on the basis that the knot area is a loss of area of the piece, regardless of the condition of the **knot**. That is, the **knot** is treated as if it were a hole.

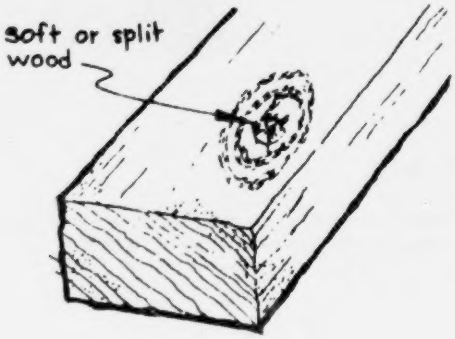
NOTE : The acceptability of **knots** may vary between the three general species — eucalyptus, radiata pine and oregon. The timber grader should check the standard for the species before grading is commenced.



Sound Tight Knot

SOUND TIGHT KNOTS

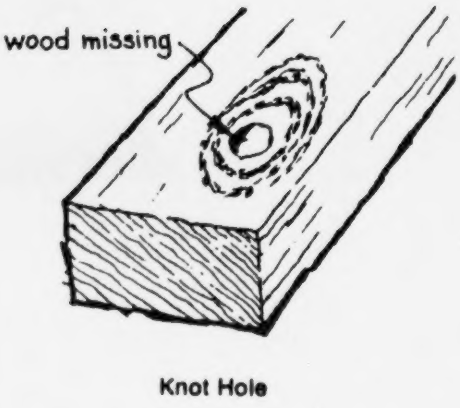
The material contained within a **sound tight knot** is hard, free from decay and firmly fixed within the piece of timber.



Unsound Knot

UNSOUND KNOTS

The material contained within an **unsound knot** may be decayed, or softer than the surrounding material of the piece. It may also be chipped, split or loose.

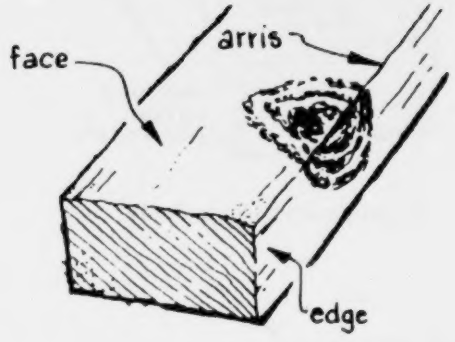


Knot Hole

KNOT HOLES

Knot holes occur where the material originally contained within the knot has been dislodged from the piece of timber.

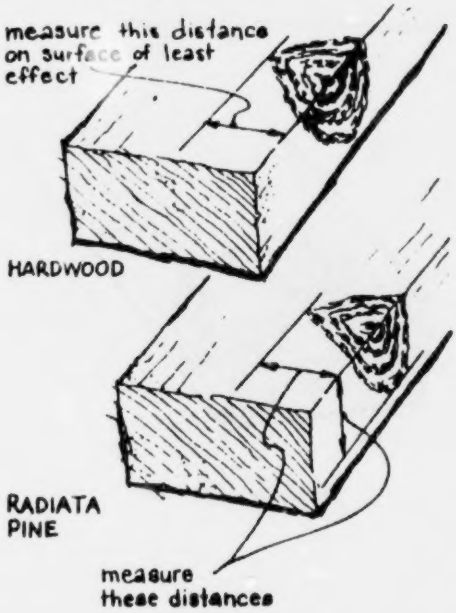
NOTE : Sloping grain surrounding a knot may be ignored. The effect of this is taken into consideration when the standards for knot acceptability are determined.


face
arris
edge

Arris Knot

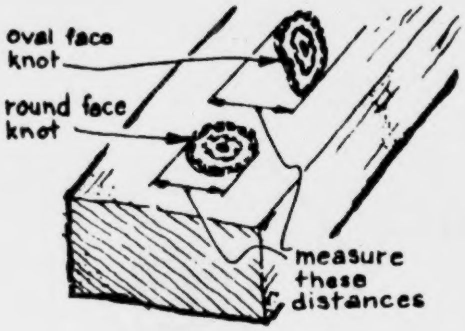
● ARRIS KNOTS

Arris knots are knots which extend across a face and an edge of a piece of timber. They include a section of the arris in their area.


measure this distance on surface of least effect
HARDWOOD
RADIATA PINE
measure these distances

Measurement of Arris Knots

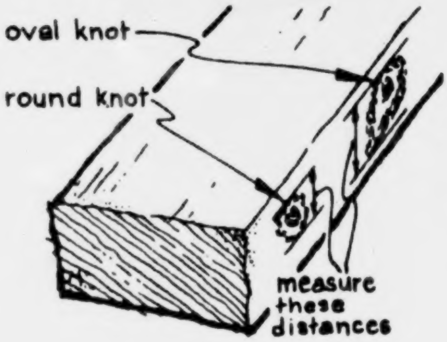
Arris knots in hardwood are measured by determining their width as for face or edge knots but on the face or edge of least effect. Where they occur in oregon or radiata pine the width of the **arris knot** is measured on the face and the edge. These two measurements are then multiplied together and the figure obtained related to the cross-sectional area of the piece to determine acceptability.


oval face knot
round face knot
measure these distances

Measurement of Face Knots

FACE KNOTS (Round or Oval)

Face knots are knots or knot holes which occur on the face of a piece of timber. Measure **face knots** by noting their greatest width measured across the face of the piece. This measurement, when related to the face width, determines the acceptability of the knot.

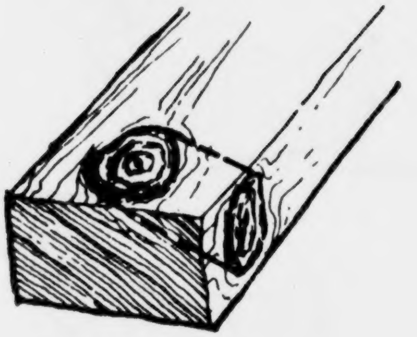

oval knot
round knot
measure these distances

Measurement of Edge Knots

EDGE KNOTS (Round or Oval)

Edge knots are similar to face knots. They occur on the edge of a piece of timber.

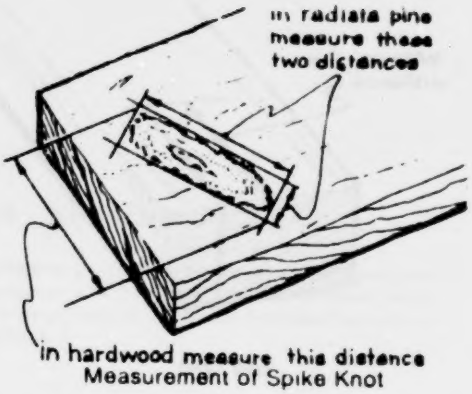
Edge knots are measured in the same way as face knots and their width in relation to the width of the edge determines their acceptability.



Through Knot

THROUGH KNOTS

Through knots are knots extending through a piece of timber between an edge and a face. The diameter of the knot is measured on the face or edge of greatest effect.


in radiata pine measure these two distances
in hardwood measure this distance

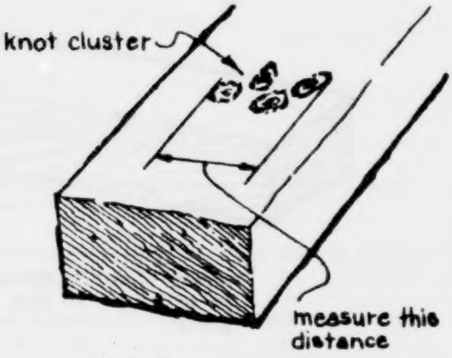
Measurement of Spike Knot

● SPIKE KNOTS

Spike knots are elongated knots with a spike shaped area. In hardwood the width between the arrises is measured. In radiata pine their size is determined as follows :

$$\frac{\text{length + width}}{2}$$

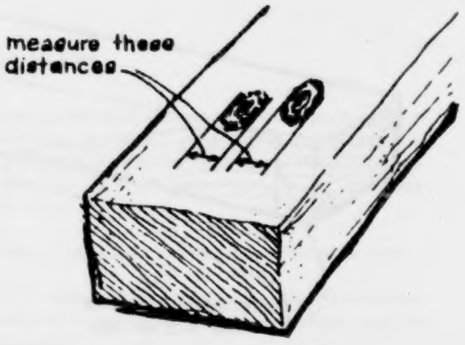
This size is then related to the width of the face or edge where the knot occurs, to determine acceptability.


knot cluster
measure this distance

Measurement of Knot Cluster

● KNOT CLUSTER

A **knot cluster** contains two or more knots within a single area of deflected grain. The overall width of the cluster is measured and related to the width of the face or edge where it occurs to determine acceptability. **Knot clusters** are usually only found in radiata pine or oregon.


measure these distances

Knot Group Measurement

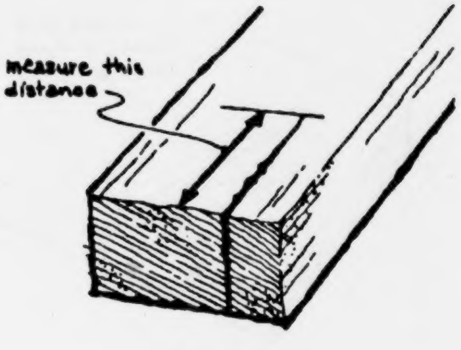
● KNOT GROUPS

A **knot group** is formed by two or more closely spaced individual knots, each knot being bounded by deflected grain. The aggregate width of the knots within the group is related to the width of the face or edge on which it occurs to determine acceptability. **Knot groups** will usually only be noted in radiata pine or oregon.

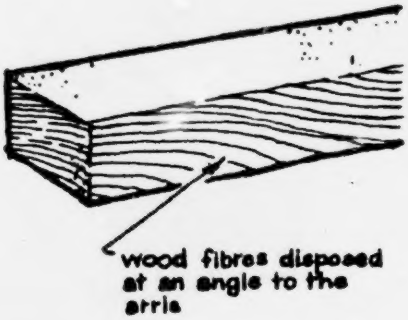
● END SPLITS

End splits are open cracks between the faces or edges of the piece. **End splits** are caused by separation of the wood fibres and extend along the piece from the ends.

Inspect each end of the piece for **end splits** and measure the length of any splits.


measure this distance

End Split

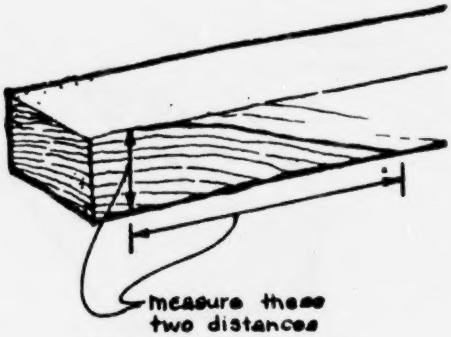

wood fibres disposed at an angle to the arris

Sloping Grain

● SLOPING GRAIN

Sloping grain is the presence of wood fibres at an angle to the arris.

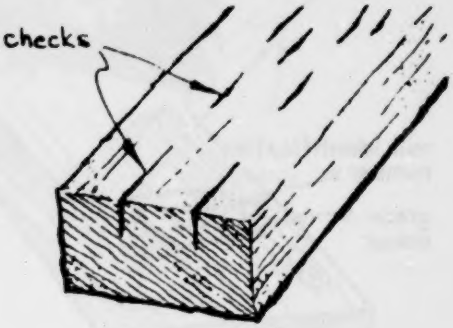
Sloping grain is a measure of the general slope of the fibres and local deviations are disregarded. It is measured and limited at that point in the length which shows the greatest slope and is expressed as a ratio such as 'slope of grain 1 in 15'.


measure these two distances

Measurement of Sloping Grain

SURFACE CHECKS

Checks are shallow cracks extending along the grain on a face or edge. **Checks** are usually short in length and do not extend between the faces or edges.



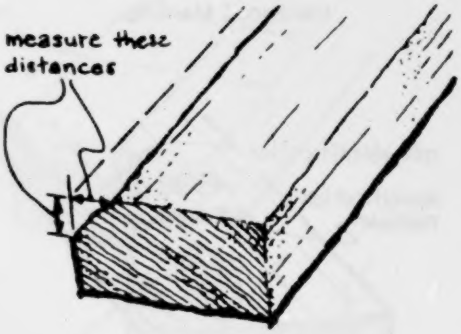
Checks

WANT

Want describes wood missing from an arris, edge or face of a piece of timber.

Want may be caused by a piece splitting off, or result from a sawing defect.

Want is measured as a face and edge width deficiency.



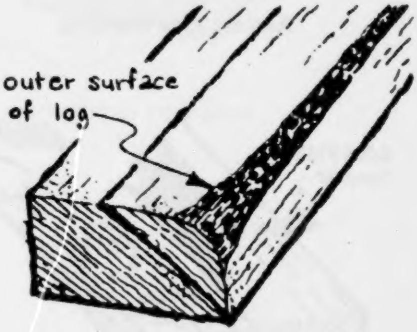
Want

WANE

Wane is the natural absence of wood from an arris or surface produced by cutting the piece from the outer **surface** of a log. The **wane area** includes portion of the log's outer surface and possibly also some bark.

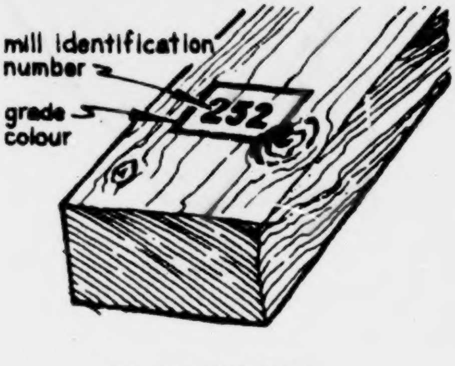
Wane is measured in the same manner as want.

NOTE **Wane** in a piece of hardwood indicates the presence of **sapwood**. The **sapwood** must also be evaluated.



Wane

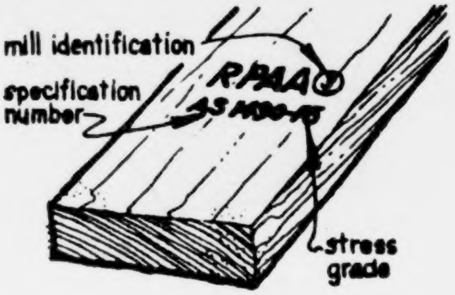
MARKING



Hardwood Marking

Timber which has been visually or mechanically stress graded, must be marked, usually by colour, with an indication of its stress grade. The marking is usually made on a face near one end of the piece. The particular mark to be applied is dictated by the prevailing standards. It may include other information to indicate species, and a mill identification code number.

These marks are noted during examination of building frames by building inspectors.



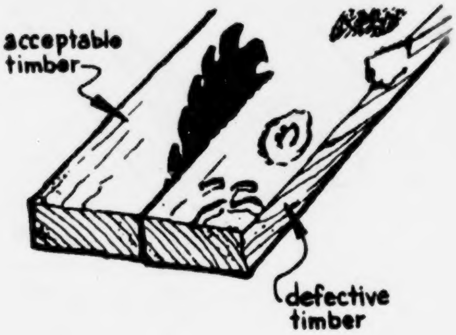
Radiata Pine Marking

REWORKING

Timber which failed to meet requirements during stress grading must be re-worked whenever possible. The grader should indicate what type of re-work is required, and note this in accordance with mill practice.

NOTE: When considering re-work the grader must have a clear understanding of the permissible extent of a defect within the grade. For example, in the case of an **end split** which exceeds the length limit imposed by the prevailing standard it would be wasteful to dock the entire portion of the length of timber affected by the **end split**. Only that length of the split which exceeds the grade limit should be docked. With practice, a competent grader is able to economically specify re-work of all defects in this manner, and keep timber wastage to an absolute minimum.

There are two main types of re-work — ripping and docking.

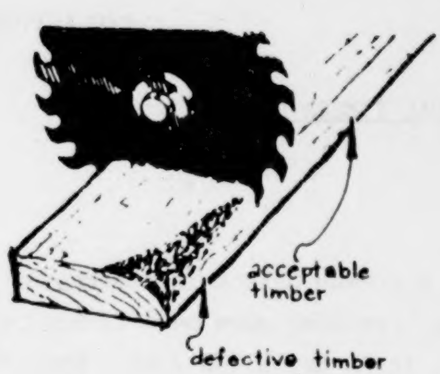


Ripping

● RIPPING

Where a defect such as **sapwood** extends along a major portion of the length of a piece but only across part of its width, it may be possible to rip the length of timber, remove the defective portion, and produce a narrower piece which meets grade requirements.

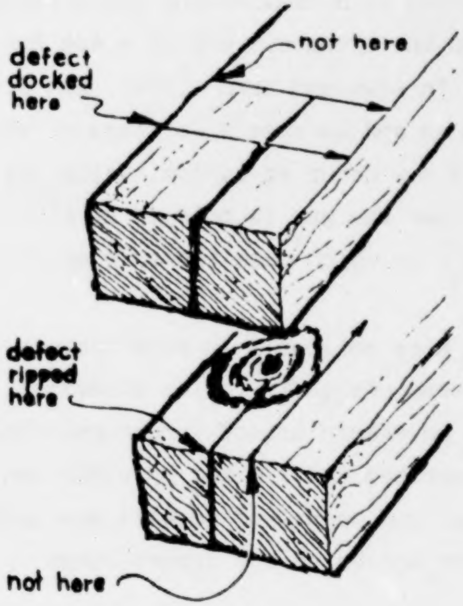
To judge if ripping is feasible the grader must decide whether the sound portion of the piece can be ripped to a standard size.



Docking

● DOCKING

Where a defect affects only a portion of the length of a reject piece of timber, for example an **end split**, **gum pocket** or **knot**, it may be possible to dock the defective portion to produce one or more shorter lengths of timber which meet grade requirements. Shorter lengths may be utilised as noggins, or finger jointed where permitted.



Re-work Within Tolerance

VISUAL JUDGEMENT

Many of the defects described require measurements to be taken during evaluation. When the grader has had some practical experience it should be possible to judge most defects accurately by eye without measurement. The attainment of this ability should be the goal of the trainee.

NOTE: The information and diagrams contained in the previous pages were reproduced from the 'Visual Stress Grading Manual' by courtesy of the Timber Promotion Council, Victoria, Australia.

EXPLANATORY NOTES

PRINCIPAL TIMBER DEFECTS

GUM

Gum is a common defect in Australian hardwoods. It is formed in the growing tree when some part is injured by fire, insects or mechanical agency. It is deposited like a shield to cover the injury. The gum attributable to fire may form around the growth ring or a substantial part of the ring. That attributable to insects and mechanical agency may be limited to a cone around the affected position. Gum rings may be evident on the cross-cut end of a log but there may be few other external signs. In sawn and hewn timber, gum becomes evident as veins on quarter-cut faces and as smears, streaks or splashes on backsawn faces, and gum pockets may occur at random. Veins may be distinguishable as 'tight', in which case the gum is bridged at close intervals by woody tissue, or 'loose', in which case there is evident separation of the wood elements.

Gum is more noticable in pale timbers than in dark timbers and is often considered unsightly in its predominating occurrences. Tight gum veins do not adversely affect structural timber. Gum pockets and loose veins are described and limited by width and length. It is important to remember that gum veins and pockets are measured radially for width, that is at right angles to the growth rings.

KNOTS

Knots are a section of the original branch of the tree. They are referred to by various names denoting the condition of the branch at the time of conversion, the shape of the knot or the condition of the knot after sawing and planing. Although knots do not occur in Australian hardwoods as often as they do in softwoods, they are an important defect to be considered.

The effect of a knot on the strength of wood in tension and bending is two-fold. Firstly, the knot itself makes no contribution to the overall strength of the wood and therefore represents a loss of cross-section, that is the knot acts as if it were a hole. Secondly, and more commonly, every knot causes a swirl or deviation of the grain acting as cross-grain, particularly if the knot is on an edge. The cross-grain can be extremely severe and its effect in reducing strength very great. In consideration of hardwood building scantling, we are concerned with two types of knot, the face knot which occurs on the wide face and the arris knot which breaks the arris and occurs on the face of the edge or on the edge only. Sizes of knots are judged on diameter, the distance between lines touching their edges drawn parallel with the edge of the piece.

SLOPING GRAIN

When timber is used for load bearing purposes, advantage is taken of the fact that timber is stronger along the grain than across it. Where theload is applied at some angle to the grain, lesser strength must be allowed for. Sawmillers have to accept practically the forest-run of logs and in many pieces of sawn timber the direction of grain is not parallel with the length of the piece and some account must be taken of the angle which the grain makes with the length. Where the angle is small the effect on strength may be ignored. Where the angle is large, the timber should not be used for structural purposes.

For many practical purposes set limits on permissible slope of grain are desirable. Various grading rules impose limits consistent with the structural sufficiency of the grade. In ladder rungs, sporting goods and tool handles the limit may be 1 in 20. Straight grain is important in barrel staves and intimber that is to be bent after steaming.

Detection of Sloping Grain

Sloping grain is not always easily detected. Growth rings are not reliable indicators of the grain slope. In fact, timber may easily be sawn from a spiral grain log in such a way that the growth ring pattern on each face appears to be parallel with the length of the piece.

For detecting sloping grain there are three principal methods:

1. To observe the direction of seasoning checks which are usually evident in sawn timber; these checks follow the grain and its direction can be found accurately by examination.

2. Split a portion off the piece (splits follow the grain) or lift a small sliver from the piece with a knife or chisel and note the direction in which the wood splinters strip.

3. By using a scribe.

CSIRO Division of Building Research, Trade Circular No. 48 'Sloping Grain in Timber' gives further information on this subject and includes instructions for measuring. The above methods are for detecting sloping grain if the grader is in doubt. Much of the sloping grain encountered is recognisable to people used to handling timber. Remember, sloping grain associated with knots does not have to be considered as a separate defect, its effect is taken into account by the permissible size for knots.

INSECT ATTACK

Timber both green and dry is subject to attack by a number of insects. The mode of action of the insect is physically to consume a portion of the timber and evidence of the attack becomes apparent in sawn timber as holes of various sizes and shapes. Experience with a particular species of timber usually provides a reliable index of what can be tolerated from a particular type of insect attack and most specifications permit minor insect damage. Holes are limited usually by diameter and by number permissible in a stated area. Severe insect attack in some instances may accompany fungal damage and, therefore, care should be exercised in assessing insect damage.

Pinhole Borers

Pinhole borers attack some damaged or suppressed trees and attack some freshly felled logs. The attacks are common in Australian hardwoods. Fortunately, the effect on strength is negligible except in rare cases. Eggs are laids by adult beetles in the green log in the forest and the larvae tunnel into the log, forming generally straight holes about 1.5 mm in diameter. Attack can only occur when the wood is green and ceases as soon as the timber is partly seasoned. The attacks are generally well scattered, but clusters may occur as galleries and where they do so particular care must be taken to ensure that they have not weakened the piece of timber.

Pinhole borers are known also as Ambrosia beetles since the larvae feed on a fungus called Ambrosia which grows on the walls of the tunnels in the wood. This fungal growth ceases as soon as the moisture content is reduced by drying of the timber. Some staining around the holes is sometimes noticed but this stained wood is not weakened.

Where the holes are scattered, the effect of pinhole attack on strength and stiffness is negligible, and in most cases appearance is not seriously affected. Where extreme clustering occurs strength will be reduced and specifications provide against clustering likely to impair strength.

Grub Holes

Other forms of insect attack which may be noted in sawn or hewn timber may be caused by beetles called longicorn, bostrychid, jewel or others. The weakening effect is roughly proportional to the degree of attack, affected timber is graded accordingly. The holes larger than 6 mm are generally well scattered and are considered in the grading rules the same as for knots.

Lyctus Attack

The lyctus beetle attacks the sapwood only of timber that contains pores and certain loadings of starch. Its effect is comletely to destroy the sapwood in susceptible species and, where the sapwood is wide, lyctus may consequently cause substantial weakening. For this reason, the amount of susceptible sapwood is limited in house framing timbers.

With most timbers there is little loss of strength possible through lyctus attack on structural pieces of large dimensions. Where the cross-section involved is small such as in studs, braces, tiling battens or small mouldings, the major portion could be vulnerable. The introduction of the Timber Framing Code could increase the problems with regard to house framing timbers as smaller sections will be used. Sapwood can be immunised against lyctus, and immunisation is required under state law in New South Wales and Queensland. Treated sapwood is in every way equivalent in use to normal true-wood.

Termite Attack

Termite attack may range from insignificant surface runs which have no appreciable weakening effect to complete destruction of the interior of a piece of wood leading to failure. The termites preferentially attack near the pith and they may be present for some years before the attack in large sections is detrimental or for some years before a pole is seriously weakened. For No. 3 Structural Grade, the formula states that termites must be on the surface only and slight. As previously stated, insignificant surface runs are permitted but care must be taken to ensure that the attack is on the surface only.

BRITTLE HEART

This defect is common in several eucalypts and is believed to have its origin in the growth stresses developed within the growing tree. It may, however, be fungal in origin. It is confined to the heart and the immediately surrounding area. The affected area may be relatively small, as in most eucalypts, or fairly extensive, as in the case of large overmature trees. Brittle heart timber is usually not detectable by simple visual examination as its appearance may be similar to sound material in every way. In some cases, brittle heart may be detected by breaking a small piece of timber and noting whether the fracture is carrotty, or by prizing up with the point of a pen knife small splinters from the surface. Brittle heart timber snaps over the point whereas sound timber tends to lift off in a long splinter.

Tests show that although the impact strength of brittle heart is very low, its static strength properties are not appreciably less than those with adjacent unaffected wood unless there is any obvious decay present. Brittle heart is low in shock resistance and durability. For framing timbers, brittle heart is completely excluded by the grading rules for all timbers with an end section of less than 175 x 175 mm.

DECAY

Decayed wood has very little strength and is particularly low in toughness. As the full extent of the decay is difficult to estimate or define and as the decay may, under adverse conditions, extend after erection, timber showing decay should not be used for purposes where strength is required.

CHECKS, SHAKES AND SPLITS

Checks show as crevices, while shakes occur around or across the growth rings. Checks are caused by unequal shrinkage in the radial, tangential or longitudinal directions. When the outside of a piece dries much more rapidly than the inside, the faces contract while the core remains unaltered, thus causing checks to appear on the face. As the interior dries the checks may close and become invisible, but nevertheless checked wood does not re-unite and consequently remains weakened.

End checking is due to local drying at the ends of logs or timber where stresses set up splits along the medullary rays, which are naturally weak. They too may close as the timber dries, but nevertheless fibres may continue to be separated and, therefore, strength is reduced.

Ring and star shakes often occur in the ends of hardwood logs after cross-cutting. They may be due to drying of the ends but the principal cause is the relief of growth stresses in the tree by cross-cutting.

Splits may be eliminated or reduced in the course of sawmilling.

WARPING

Distortions may develop in timber for a variety of reasons and the reputation of timber as a commercial material is adversely affected by them. The principal kinds are bow, cup, spring and twist. They are generally more evident in dry than green timber. However, bow and spring often occur in the course of sawmilling and some occurrences can be attributed to the uneven distribution of stress in logs. They can increase the problems of controlling dimensions during sawmilling and severe spring in particular causes economic losses. They can be accentuated during seasoning but can also be controlled or reduced by suitably placing pieces in seasoning stacks and carrying out the seasoning under weights and restraints associated with steaming treatments. Spring is particularly critical in studs used in light timber framing.

Cup and twist develop mainly during seasoning due to differential shrinkage in the several directions relative to the growth rings.

Warp can be limited to stated amounts per unit of length for bow and spring, per unit of width for cup, and per unit of both length and width for twist.

WANE

Imperfect sawing sometimes results in the retention on part of the corner, or part of a face or an edge, of a portion of the original log surface. This is called wane. It is accepted on structural pieces when the actual section meets the strength requirements or the fixing requirements for the piece.

ACCEPTABILITY OF DEFECTS

In the course of drafting grading rules for Australian timbers, the significant characteristics of timber have been realistically appreciated. The seriousness of any irregularity, imperfection, blemish, fault or defect is judged primarily with respect to its influence upon:

1. Appearance
2. Strength

The intended use determines the relative importance of the one consideration as against the other.

Some defects are unsightly, some reduce strength, and some affect other aspects of serviceability. As timber is used for many different purposes, there are many criteria of acceptability and the significance of a defect of certain type and size in some uses changes in other uses.

The influence of the above features or defects on appearance is regarded as a matter of opinion. The influence of defects on strength is a technical question that needs to be answered by controlled tests.

There is nothing highly technical involved in deciding limits for grades based on appearance factors. A buyer requires the highest standard possible. The seller points out that, with the best of intentions and deliberate effort to eliminate certain faults, timber will not be perfect. While there may be difficulty in attaining unanimous agreement on the significance of an irregularity, consensus of opinion, or compromise between some conflicting attitudes of producers and users, has eventually to become basic to the preparation of a specification for grading rules. For panelling, furniture and other woodwork likely to be finished with clear coatings, irregularities which contrast unduly with the main appearance of the wood may be regarded as defects. Thus, black stains on a pale background, or white streaks in a dark surface, holes, broken or decayed knots, torn grain and others may consistently be regarded as faults by many users so that they are unacceptable in timber used for the exacting purposes mentioned.

The influence of defects on the strength has been investigated in considerable detail. It is found that slope of grain, knots, splits, decay and holes are the principal causes of loss of strength. The effect of these types in various sizes and positions have been determined. From the investigations, it is possible to say that, providing grain does not exceed a specified slope, pieces will have strength not less than a stated percentage of straight grain timbers; that provided the size of a knot does not exceed a certain diameter, the piece will have a strength not less than that of a related percentage of knot free timber; that provided the wane does not result in more than a certain loss of section, a piece will have not less than a certain percentage of the strength of a complete rectangular section; and that other main defects have other determinate influences.

Accordingly, from a knowledge of relations between defects and strength it becomes possible to state the limits for type and size which will ensure that pieces sorted to these limits will not be lower in strength than a specific percentage of the strength of defect free timber. Thus the basis of grading rules for structural timber emerges.

The effects of malformations and damage to trees revealed as defects in products are more or less objectionable to different individuals in connection with particular uses. What may be considered a defect for structural purposes may be accepted as a feature where appearance is the only consideration. Usually, if the defect is small in size and isolated, it may be accepted without question. However, what is small in the opinion of a seller may not be regarded as small by a purchaser. There needs to be some clarification of terms or some specific descriptions. Especially it is desirable for any statement to have only one meaning to all persons concerned.

For these reasons, there have been conscientious efforts made to write grading rules and specifications precisely. The Standards Association of Australia has attended to this matter, bringing together producers, consumers and others interested in an item being considered in order to discuss representative points of view ahead of standardisation. The Association is a medium of publication for the standards endorsed by the interests affected.

Grading Rules: Hardwood F8

GREEN IS THE PRESCRIBED COLOR FOR THE BRAND MARK TO DENOTE F8 STRESS GRADE TIMBER

SIMPLIFIED INFORMATION FOR CONSTRUCTION TIMBER (structural) BASED ON AS2082-1979.

General: Each piece of timber of structural grade No. 3 shall be free from compression failures and other fractures.

Size: Each piece shall be sized with the maximum tolerance of 2mm for this season timber.

NOTE: Each parcel of timber to this grade shall contain a reasonable distribution of material ranging from the lower limit detailed below to material which just falls short of the next highest grade.

IMPERFECTIONS		MAXIMUM PERMISSIBLE LIMITS
CHECKS	Internal	Not to exceed ½ thickness
	Surface	Unlimited
KNOTS	Sound	
	Oval	Not to exceed 1/3 width
	Round	Not to exceed 1/3 thickness
	Arris Spike	Not to exceed 1/3 on least face or edge
	Loose Knots - Holes	Same as a Sound Knot
BORERS HOLES	Over 3mm	Over 3mm or where distance between is less than 3mm assess as Sound Knot
	Under 3mm	Unlimited, provided distance between is minimum of twice width of hole
GUM VEINS	Tight Loose	Unlimited
Loose Gum and Shakes	Width	Not to exceed 3mm 1/4 length aggregate
	Length	Not to go through one surface to another
GUM POCKETS	Length	Not to exceed 3 times width or 300mm maximum
	Width	
	- 1 surface	1/2 width or 25mm maximum
	- through	1/3 width or 20mm maximum
BOW		35mm in a piece of 3m long x 38mm thick. (refer to tables)
SPRING		14mm in a piece of 3m long x 100mm wide. (refer to tables)
TWIST		1mm per 10mm of width in pieces. 3m long x 38 mm thick (refer to tables)
CUPPING		1mm in 50mm of width
SLOPE OF GRAIN		1 in 8
DECAY/TERMITES		On surface only and slight
WANT/WANT/SAPWOOD		1/4 of cross section (aggregate) 1/3 thickness except for 1/3 length where it may extend to the full width of the thickness
END SPLITS		Aggregate length at each end. 1½ times width or 150mm maximum
HEART		Not permitted in section under 175mm x 175mm

This information sheet has been prepared as a ready reference guide. For complete details of grading rules, reference should be made to AS2082-1979.

Grading Rules: Radiata Pine F5

BLACK IS THE PRESCRIBED COLOR FOR THE BRAND MARK TO DENOTE F5 STRESS GRADE TIMBER

SIMPLIFIED INFORMATION FOR CONSTRUCTION TIMBER (structural) BASED ON AS 1490-1973.

General: Each piece of timber shall be free from decay, shakes, splits and fractures.

Size: Each piece shall be of actual dimensions with a maximum tolerance of 3mm OVERSIZE only.

Note: Each parcel of timber to this grade shall contain a reasonable distrubution of timber ranging from the lower limits of this grade to just short of the next highest grade.

IMPERFECTIONS		MAXIMUM PERMISSIBLE LIMITS
CHECKS		Maximum 600mm long
KNOTS	Face	
	- Centre Margin	4/5 (80%) of face width
	- Face Knot	3/5 (60%) of face width
	Margin	
	- Margin	(10%) of face width
	- Knot	7/16 (43%) of face width
	Edge	2/3 (66%) of thickness
	Arris	7/16 (43%) area of section
	Holes	Measure as a knot
RESIN POCKETS		
	Length	Maximum 150mm
	Width	Maximum 12mm
STAIN		Unlimited
HEART/PITH		1. Only in sizes 240mm and over 2. Must be in middle 1/3
SLOPING GRAIN		1 in 5
WANE/WANT		1. 1/4 (25%) of area of section 2. Maximum 1/4 of any surface
BOW		40mm in a piece 3m in length and 35mm thick (see tables)
SPRING		16mm in a piece 3m in length and 90mm wide (see tables)
TWIST		3mm for every 10mm of surface width (see tables) e.g. 27mm in a piece 90 x 35mm. 3m in length.

This information sheet has been prepared as a ready reference guide. For complete detail of grading rules, reference should be made to AS 1490-1973.

Grading Rules: OREGON (Douglas fir) F5.

Black is the prescribed colour for the brand mark to denote F5 stress-graded timber ●

SIMPLIFIED INFORMATION, FOR CONSTRUCTION TIMBER (structural), BASED ON AS 2440-1981.

General: Timber shall be truly sawn and free from decay and compression failure. Damage resulting from the use of hooks, dogs, slings and the like shall be accepted provided the effect of the damage is not more serious than that of a permitted imperfection.

Size: The minimum off-the-saw dimension for this unseasoned timber must not be more than 4 mm under the nominal size.

The following MAXIMUM imperfections shall be permitted on the worst face and edge:

Note: Each parcel of timber to this grade shall contain a reasonable distribution of material ranging from the lower limit detailed below to material which just falls short of the next highest grade.

IMPERFECTIONS (General Description)	MAXIMUM PERMISSIBLE LIMITS
CHECKS	Permitted
FACE KNOTS	5/8 W on Centre Line Elsewhere Proportional (See note over)
MARGIN KNOTS	3/8 W
THROUGH KNOTS	As Face or Margin Knots (Measure only on the face of the piece)
EDGE KNOTS	3/5 T
ARRIS KNOTS	X Sectional Area = 3/8 W x T Useful working guide: 1/2 x 3/4 = 3/8 Therefore maximum permissible arris knot size would be: 1/2 width of face by 3/4 thickness of edge or 3/4 width of face by 1/2 thickness of edge
HOLES AND LOOSE KNOTS	Measure as sound knots in same position
RESIN STREAKS	Permitted
RESIN POCKETS	Permitted
RESIN BLISTERS	Permitted
WANE AND/OR WANT	$\frac{W + T}{3}$ (MAX $\frac{W}{2}$, $\frac{T}{4}$, $\frac{1}{3}$ agg. L)
SAPWOOD	Permitted
END SPLITS	Length: Twice width of piece (at both ends) 2 x W
SHAKES	Centre face 5/8 T Elsewhere 3/4 T
STAIN/DISCOLORATION	Permitted
SOUND HEART	Permitted
SLOPE OF GRAIN	1 in 6
SPRING	25 mm in pieces 3.6 m long x 100 mm wide OR equivalent curvature in other lengths and widths. (Refer 'Spring and Bow' Tables)
BOW	50 mm in pieces 3.6 m long x 50 mm thick OR equivalent curvature in other lengths and thicknesses. (Refer 'Spring and Bow' Tables)
TWIST	1 mm per 10 mm of width in pieces 3.6 m long x 50 mm thick OR equivalent in other lengths and thicknesses.

This information sheet has been prepared as a ready-reference guide. For complete details of the Grading Rules, reference should be made to Australian Standard

Grading Rules: OREGON (Douglas fir) F7

Blue is the prescribed colour for the brand mark to denote F7 stress-graded timber ●

SIMPLIFIED INFORMATION, FOR CONSTRUCTION TIMBER (structural), BASED ON AS 2440-1981.

General: Timber shall be truly sawn and free from decay and compression failure. Damage resulting from the use of hooks, dogs, slings and the like shall be accepted provided the effect of the damage is not more serious than that of a permitted imperfection.

Size: The minimum off-the-saw dimension for this unseasoned timber must not be more than 4 mm under the nominal size.

The following MAXIMUM imperfections shall be permitted on the worst face and edge:

Note: Each parcel of timber to this grade shall contain a reasonable distribution of material ranging from the lower limit detailed below to material which just falls short of the next highest grade.

IMPERFECTIONS (General Description)	MAXIMUM PERMISSIBLE LIMITS
CHECKS	Permitted
FACE KNOTS	1/2 W on Centre Line Elsewhere Proportional
MARGIN KNOTS	1/3 W
THROUGH KNOTS	Measure only on the face of the piece
EDGE KNOTS	1/2 T
ARRIS KNOTS	X Sectional Area = 1/3 W x T Useful working guide: 1/2 x 2/3 = 1/3 Therefore maximum permissible arris knot size would be: 1/2 width of face by 2/3 thickness of edge or 2/3 width of face by 1/2 thickness of edge
HOLES & LOOSE KNOTS	Measure as sound knots in same position
RESIN STREAKS	Up to 375 mm and 1/3 Length in agg.
RESIN POCKETS	Up to 375 mm and 1/3 Length in agg.
RESIN BLISTERS	Up to 375 mm and 1/3 Length in agg.
WANE AND/OR WANT	$\frac{W + T}{6}$ (MAX $\frac{W}{4}$, $\frac{T}{4}$, $\frac{1}{4}$ agg. L)
SAPWOOD	Permitted
END SPLITS	Length: Width of piece (at both ends)
SHAKES	Centre Face 1/2 T Elsewhere 3/4 T
STAIN/DISCOLORATION	Permitted
SOUND HEART	Permitted
SLOPE OF GRAIN	1 in 8
SPRING	19 mm in pieces 3.6 m long x 100 mm wide OR equivalent curvature in other lengths and widths. (Refer 'Spring & Bow' Tables)
BOW	38 mm in pieces 3.6 m long x 50 mm thick OR equivalent curvature in other lengths and thicknesses. (Refer 'Spring & Bow' Tables).
TWIST	1 mm per 10 mm of width in pieces 3.6 m long x 50 mm thick OR equivalent in other lengths and thicknesses.

This information sheet has been prepared as a ready-reference guide. For complete details of the Grading Rules, reference should be made to Australian Standard

Grading Rules: OREGON (Douglas fir)

SELECT DRESSING GRADE

SIMPLIFIED INFORMATION, FOR CONSTRUCTION TIMBER (structural), BASED ON AS 2440-1981.

General: Timber shall be truly sawn and free from decay and compression failure. Damage resulting from the use of hooks, dogs, slings and the like shall be accepted provided the effect of the damage is not more serious than that of a permitted imperfection.

Size: The minimum off-the-saw dimension for this unseasoned timber must not be more than 4mm under the nominal size.

The following **MAXIMUM** imperfections shall be permitted on the worst face and edge.

Select dressing grade timber is intended to provide material for finishing and joinery purposes where better appearance is the primary consideration. The timber shall be truly sawn and free from decay. The following imperfections shall be permitted:

IMPERFECTIONS (General Description)	MAXIMUM PERMISSIBLE LIMITS
CHECKS	150mm long
FACE KNOTS MARGIN KNOTS THROUGH KNOTS EDGE KNOTS ARRIS KNOTS HOLES OR LOOSE KNOTS	Not more than 3 in 3.6m length x 200mm width. A proportional Number of Knots shall be Permitted on other Lengths and Widths Providing the Knot does not Exceed 1/8W or 1/3T or 25mm, whichever is least. 1/2 Permitted for Sound Tight Knots or 10mm, whichever is least.
RESIN STREAKS RESIN POCKETS RESIN BLISTERS	150mm Long individually 150mm Long individually 150mm Long individually
WANE	Not permitted
SAPWOOD	Permitted
END SPLITS	1/2 W
SHAKES	Not permitted
STAIN	Not Permitted
SOUND HEART (PITH)	Not permitted
SLOPE OF GRAIN	Permitted
SPRING	38mm in 3.6 Length
BOW	38mm in 3.6 Length
TWIST	Not permitted
DISCOLOURATION	Permitted

REFERENCE LIST

1 Australian Standard AS2082 — 1979 — Visually Stress Graded Hardwood for Structural Purposes.

2 Australian Standard AS1490 — 1973 — Visually Stress Graded Radiata Pine for Structural Purposes.

3. Australian Standard AS2440 — 1981 — Sawn Douglas Fir (Oregon) and Sawn Western Hemlock (Canada Pine)

4. Sloping Grain in Timber — C.S.I.R.O. Trade Circular No. 48.

REVIEW OF TIMBER STRENGTH GROUPING SYSTEMS

William G. Keating[1]

1. INTRODUCTION

To the casual observer it must seem strange that in some countries, particularly those where forests are an obvious major natural resource, the structural use of timber lags well behind the level attained by other construction materials. There are probably many reasons based on economic, technical and even cultural aspects why this is the case. One reason surely must be the problem involved in presenting structural data to the end-user in an appropriate fashion whenever there is a multiplicity of species involved. A technique devised to minimize this problem is grouping.

To give just a few examples of the magnitude of the problem, Pong Sono (1974) has listed approximately 200 species of merchantable timber in Thailand, Espiloy (1978) notes that in the Philippines there are over 3000 timber species of which several hundred are probably potentially merchantable. In Australia, while there are about 80 species used extensively, more than 500 species have been classified for structural use (Standards Association of Australia 1979a). Many of these species are sold in mixtures because of the practical difficulties associated with their identification and segregation, but strength grouping is able to cope with this requirement (Leicester and Keating 1982).

2. BASIS OF GROUPING

Essentially grouping for structural purposes means the creation of a preferably small set of hypothetical species so that any timber may be classified within this set and considered as equivalent to one of the hypothetical species.

From a survey of the literature it would appear that many countries have either adopted the Australian system of strength grouping as described by Pearson (1965) and Kloot (1973) or have used it as the basis for developing their own systems. Some of the countries are Kenya, Tanzania, Nigeria, Papua New Guidea, Fiji, Samoa and Solomon Islands. In addition, the United Nations Industrial Development Organization has used the technique in the development of the design of a low-cost modular prefabricated wooden bridge. Of course there are many other systems in use, but most of the well known ones such as those used in Northern America are, in the main, concerned with a comparatively small number of softwood species.

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3. MOTIVATION FOR GROUPING

The degree of motivation for adopting a classification system based on structural properties varies directly with the number of species that are required to be accommodated. Without grouping, the problems involved are most obvious when it comes to publishing design information. Even if the data on a large number of species from a particular country were available, it is often not feasible to publish the relevant design information in a readily accessible form. This is where the use of grouping techniques makes such data presentation much easier.

The area of building regulations is one where grouping introduces advantages that are of particular value (Leicester 1981). Besides the obvious simplification regulations written in terms of groups rather than individual species have tables of design properties incorporated within them that remain fixed. This means that no major change is involved should a new timber be introduced on to the market or an existing one be reassessed. In Australia the SAA Timber Framing Code AS 1684 (Standards Association of Australia 1979b) through a limited set of tables manages to present spans and sizes of all the timber framing members required in domestic housing construction, applicable to all grades for several hundred species or species mixtures, in a most convenient format.

Even in the case where a single species dominates the timber construction scene, grouping in relation to building regulations is advantageous. The structural properties of populations of timber taken from the same species, particularly with plantation timbers, can vary from one forest location to the next and can also vary with forest age and silvicultural practices. Transferring a species, or the production from one area, from one group to another is not nearly as complicated as promulgating a new or additional set of design stresses (Leicester, 1981).

Internationally an agreed grouping technique could help timber utilization generally and have special relevance to the structural timber trade. The UNIDO Bridge Project mentioned previously is a good example of grouping applied to the world situation as the set of design standards based on eight strength classes is directly applicable for almost any timber in the world. It is not difficult to envisage how other examples of technology transfer in the form of timber design codes and manuals would be possible if an agreed or compatible grouping system for structural timber was in general use. The grouping technique has the following advantages:

1. Building regulations are concerned with only limited sets of design parameters;
2. Marketing of structural timber is easier as it is carried out in terms of structural properties rather than by nomination of the species and grading methods;
3. More flexibility is available to the supplier as the range of species is much wider;
4. The entry of new lesser-known species onto the market is facilitated;
5. Trade, both internal and international, in structural timber is simplified;
6. Technology transfer in the form of timber design codes and manuals, is easier;
7. It is much less expensive in time and material to place a species in a group than it is to develop individual working stresses; and
8. It is possible to group a species, albeit conservatively, based on density measurements alone.

4. EXISTING STRENGTH GROUPING SYSTEMS

It is proposed to discuss strength grouping systems in a few selected countries.

(a) Australia

Strength grouping in Australia has been in operation now for more than forty years. Langlands and Thomas (1939), in their Handbook of Structural Timber Design, proposed for Australian conditions four strength groups. A species was placed in a group according to its species mean values as determined from standard tests on small clear specimens. These strength groups were established when there was little information available about the properties of most Australian species and their successful use was possible only because the limits were not closely defined (Pearson 1965).

The impetus at that time to establish strength groups came, as it does now, from the need to cope with a large number of species, many of which are difficult to identify and many are also marketed as mixtures.

The original Australian strength grouping system was revised and expanded as has been explained by Pearson (1965) and Kloot (1973). Prior to the expansion of the strength groups, made necessary to cope with new information and new species, Pearson developed a set of working stresses that has now become the basis for a strength classification system.

Working back from the set of working stresses, it was then possible to develop the appropriate strength groups. This process is the reverse of the usual procedure for deriving working stresses for an individual species allowing for duration of load, accidental overloads and estimating the 1% probability point.

In the development of this set of stresses, Pearson reported that three decisions were required. Firstly, it was necessary to decide whether the stresses should be in arithmetic or geometric progression. Secondly, a compromise was required on the magnitude of the differences between successive stresses in order to achieve a satisfactory balance between simplicity associated with having only a few groups and the greater efficiency associated with numerous groups. Finally, the actual value of the stresses had to be decided.

Cooper (1953) had shown the merits of a geometric series for working stresses and such a choice had also been recommended by the International Organization for Standardization (ISO) and the Food and Agriculture Organization (FAO). Accordingly, such a choice was made using a preferred number series with adjacent terms chosen in the ratio of 1.25 to 1 for Modulus of Rupture. This was judged to be the appropriate compromise between simplicity and preciseness. Also, as appeared certain, the Australian visual grading rules then being developed would probably have differences between grades also of 25%. The range of the values chosen was such that it covered all the species likely to be used structurally in Australia.

Using the set of values decided upon as the basic working stresses in bending, the values of the other properties were determined from regression equations.

From this technique has developed Table 1, which is the basis of the current Australian strength classification system.

As described above, the species mean values for clear material for each strength group for the critical properties were developed for green and dry timber and are shown in Tables 2 and 3 respectively.

TABLE 1
DESIGN PROPERTIES FOR SAWN TIMBER, ROUND POLES AND PLYWOOD

Stress* grade	Basic bending strength (MPa)**	Basic tension strength (MPa)	Basic compression strength (MPa)	Modulus of elasticity (MPa)
F34	34.5	20.7	26.0	21 500
F27	27.5	16.5	20.5	18 500
F22	22.0	13.2	16.5	16 000
F17	17.0	10.2	13.0	14 000
F14	14.0	8.4	10.2	12 500
F11	11.0	6.6	8.4	10 500
F8	8.6	5.2	6.6	9 100
F7	6.9	4.1	5.2	7 900
F5	5.5	3.3	4.1	6 900
F4	4.3	2.6	3.3	6 100
F3	3.4	2.1	2.6	5 200
F2	2.8	1.7	2.1	4 500

* The insertion of the letter F before each value in the Table introduces the concept of stress grade. Stress grade is defined as the classification of a piece of timber for structural purposes by means of either visual or mechancial grading to indicate primarily the basic working stress in bending in megapascals for purposes of design and by implication the basic working stresses for other properties normally used in engineering design. For example, a piece of timber with a stress grade of F14 resulting from a certain combination of strength group and visual grade would have a basic working stress in bending of 14 megapascals.

** These values are the result of a soft metric conversion of a preferred series of values in imperial units viz. 5000, 4000, 3200, 2500, 2000, 1600, 1250, 1000, 800, 630, 500, 400 p.s.i., readily recognisable as the R10 series.

TABLE 2
PRELIMINARY CLASSIFICATION VALUES FOR UNSEASONED* TIMBER

Property	Minimum species mean						
	S1	S2	S3	S4	S5	S6	S7
Modulus of rupture (MPa)	103	86	73	62	52	43	36
Modulus of elasticity (MPa)	16300	14200	12400	10700	9100	7900	6900
Maximum crushing strength (MPa)	52	43	36	31	26	22	18

* As measured or estimated at a moisture content above fibre saturation point.

TABLE 3
PRELIMINARY CLASSIFICATION VALUES FOR SEASONED* TIMBER

Property	Minimum species mean							
	SD1	SD2	SD3	SD4	SD5	SD6	SD7	SD8
Modulus of rupture (MPa)	150	130	110	94	78	65	55	45
Modulus of elasticity (MPa)	21500	18500	16000	14000	12500	10500	9100	7900
Maximum crashing strength (MPa)	80	70	61	54	47	41	36	30

* As measured or adjusted to a moisture content of 12 per cent

TABLE 4
RELATIONSHIP BETWEEN STRENGTH GROUP, VISUAL GRADE AND STRESS GRADE FOR GREEN TIMBER

Visual grade*		Stress grade						
Nomenclature	% strength of clear material	S1	S2	S3	S4	S5	S6	S7
Structural grade No.1	75	F27	F22	F17	F14	F11	F8	F7
Structural grade No.2	60	F22	F17	F14	F11	F8	F7	F5
Structural grade No.3	48	F17	F14	F11	F8	F7	F5	F4
Structural grade No.4	38	F14	F11	F8	F7	F5	F4	F3

* Australian Standard AS 2082-1977, Visually stess-graded hardwood for structural purposes; and AS 1648-1974, Visually stress-graded cypress pine for structural purposes.

Note the interlocking effect (diagonal line) reducing a possible 28 stress grades to 10.

TABLE 5
RELATIONSHIP BETWEEN STRENGTH GROUP, VISUAL GRADE AND STRESS GRADE FOR SEASONED TIMBER

Visual grade*		Stress grade							
Nomenclature	% Strength of clear material	SD1	SD2	SD3	SD4	SD5	SD6	SD7	SD8
Structural grade No.1	75		F34	F27	F22	F17	F14	F11	F8
Structural grade No.2	60	F34	F27	F22	F17	F14	F11	F8	F7
Structural grade No.3	48	F27	F22	F17	F14	F11	F8	F7	F5
Structural grade No.4	38	F22	F17	F14	F11	F8	F7	F5	F4

* Australian Standard AS 2082-1977, Visually stress-graded hardwood for structural purposes; AS 2099-1977, Visually stress-graded seasoned Australian grown softwood (conifers) for structural purposes (excluding radiata pine and cypress pine); AS 1490-1973, Visually stress-graded radiata pine for structural purposes; and AS 1648-1974, Visually stress-graded cypress pine for structural purposes.

By use of Tables 2 and 3, every species that had been or was capable of being properly sampled and tested by standard methods using small clear specimens may be strength grouped. Once strength grouped, commercial pieces of that species can, following visual grading, be allocated a stress grade by reference to Tables 4 and 5.

From Table 1 the appropriate design parameters may be determined.

Because of international agreement on the standard methods of test for small clear specimens, it is possible to utilise data from recognised laboratories anywhere in the world to place any species into a strength group. This has been done for 700 African (Bolza and Keating 1972), 190 South American (Berni _et al._ 1979) and 362 South-East Asian species (Keating and Bolza 1982).

One assessment that is often required in classifying a species from Tables 2 and 3 is what to do when the three properties do not all have the same classification. A conservative approach would be to assign the species to the lowest group indicated from the individual properties. This must apply for many combinations, but there are several for which raising the overall species strength group one step above the lowest assessment is deemed justified.

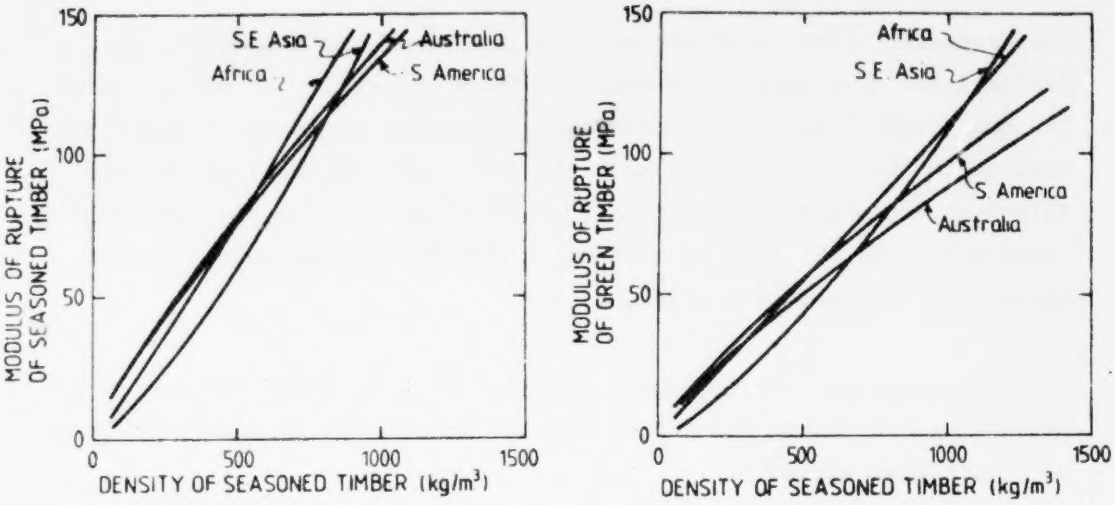
Table 6 summarises the procedure that is followed indicating that more emphasis is placed on modulus of rupture and modulus of elasticity than on compression strength.

TABLE 6
COMBINATIONS OF PRELIMINARY CLASSIFICATIONS THAT PERMIT THE OVERALL STRENGTH GROUP ASSESSMENT TO BE ONE STEP ABOVE THE LOWEST IN THE COMBINATION

Preliminary classification based on -			Assessed S or SD strength group
Modulus of rupture	Modulus of elasticity	Maximum crushing strength	
x	x	x + 1	x
x	x - 2	x - 1	x - 1
x	x + 2	x + 1	x + 1

NOTE: Strength group x - 1 is stronger than strength group x; e.g. if strength group S4 is denoted by x then strength group S3 is denoted by x - 1.

This leaves those species for which the strength data available are from less than a valid sample, assessed as a minimum of five trees, or is just not available at all. A recent examination by Leicester and Keating of the relationship between density and modulus of rupture of seasoned timber for 30 species from each of four regions around the world is indicated in Figure 1.



(iv) Modulus of rupture

(iv) Modulus of rupture

Figure 1 Regression lines for modulus of rupture versus density of seasoned timber

On the basis of this relationship, the following table was constructed to permit a classification to take place. This gives a rather conservative assessment, but at least it does allow those species with limited data to be entered into the system. In the Australian Standard MP 45-1979, Report on Strength Grouping of Timbers, species assessed in this fashion are listed with their strength groups in brackets to indicate the provisional nature of the assessment.

TABLE 7
MINIMUM AIR-DRY DENSITY VALUES FROM 5 OR MORE TREES FOR ASSIGNING SPECIES TO STRENGTH GROUPS IN THE ABSENCE OF ADEQUATE STRENGTH DATA

(a) Unseasoned Material

Strength group	S1	S2	S3	S4	S5	S6	S7
Air-dry density at 12 per cent moisture content (kg/m^3)	1180	1030	900	800	700	600	500

(b) Seasoned Material

Strength group	SD1	SD2	SD3	SD4	SD5	SD6	SD7	SD8
Air-dry density at 12 per cent moisture content (kg/m3)	1200	1080	960	840	730	620	520	420

To this point, discussion has been confined to the means of entering the strength classification system in Table 1 by way of strength grouping combined with visual grading. Direct entry into the system is also possible through machine stress grading and proof grading. Both these techniques are described in a later section.

(b) United Kingdom

Details of the system currently under discussion in the United Kingdom to be the basis of the revision of the British Standard Code of Practice on the Structual Use of Timber CP112:1967, have been given by Mettem (1981). Briefly nine strength classes have been proposed, C1 to C9, as shown in Table 8.

TABLE 8
DRY GRADE STRESSES AND MODULI OF ELASTICITY FOR STRENGTH CLASSES AS PROPOSED FOR BS 5268 : PART 2 (MPa)

Strength Class	Bending	Tension	Compression parallel	E	
				Mean	Min.
C1	2.8	2.2	3.5	6 800	4 500
C2	4.1	3.2	5.3	8 000	5 000
C3	5.3	4.2	6.8	8 800	5 800
C4	7.5	5.1	7.9	9 900	6 600
C5	10.0	6.0	8.7	10 700	7 100
C6	12.5	7.5	13.6	12 100	9 200
C7	15.6	9.5	15.7	14 900	11 400
C8	19.5	12.0	18.2	17 900	13 800
C9	24.4	15.0	21.3	21 500	16 700

The derivation of these stresses and the allocation of the various grades of those softwood species in common use in the United Kingdom to the appropriate strength classes was based on a testing program using structural sized timber. The range of species tested in this fashion did not include all those in use but for those not yet tested recourse was made to the small-clear test data and a ratio applied based on the 5th percentile results obtained from the tests on structural sized timber.

For the softwoods visually graded to BS 4978:1973 'Timber grades for Structural Use', the two visual grades, GS General Structural and SS Special Structural, cater for most of the imported softwood species have been allocated to the C3 and C4 strength classes respectively. The grade ratios (i.e. the comparison with clear strength values) for these two grades are considered to be 0.35 and 0.50 respectively in bending (Mettem 1981).

For tropical hardwoods that will also be included in BS 5268 only one grade, HS Hardwood Structural, is proposed with a grade ratio in bending of 0.67. As with the softwoods the basic stresses were based on a combination of test data from small clears and the 5th percentile values obtained from testing of structural sized timber.

(c) Philippines

In the Philippines a system has been developed that is very similar to the Australia strength grouping system in that it is based on the results of small clear tests and adopts a preferred number progression with an interval of 1.25 between the base numbers (Espiloy 1977). However it was judged that there was no need to cover the same range so only five groups have been chosen. The advantages of the grouping system according to Espiloy, are:

(i) Each member species within a class can substitute for the other, thus in a way overcome the problem of supply.

(ii) The traditional bias against the lesser-known species is easily overcome when these are grouped together with the more common species. Hence, this system will help engineers and architects familiarize themselves with alternate species by specifying that any timber within a given class may be used instead of specifying the timbers by name.

(iii) It will overcome the problem that is usually encountered in identifying sawn timber of similar physical and strength characteristics.

(iv) Grouping will simplify design and specification procedure and thus facilitate the formulation of a comprehensive building code for structures using solid wood. The grouping scheme will form a rational series that will fit closely with timber grades. With this system, only a few sets of working stresses are adequate to cover the proposed strength classes and grades of timber.

The limiting average values for classifying a species into one of the strength classes, C1 to C5, are given in Table 9.

A procedure has been developed to cover the case when the property values for a particular species do not all fall within the same strength class.

TABLE 9
MINIMUM STRENGTH-CLASS LIMITS FOR GROUPING PHILIPPINE TIMBER SPECIES

Property	Moisture condition	Class of timber				
		C1	C2	C3	C4	C5
Modulus of rupture in bending (kg/cm²)*	Green 12% MC	800 1250	630 1000	500 800	400 630	315 500
Modulus of elasticity in bending (1000 kg/cm²)	Green 12% MC	130 160	100 120	77.0 95.0	60.0 73.0	46.0 56.0
Compression parallel to grain (kg/cm²)	Green 12% MC	400 650	305 500	235 385	185 300	140 230
Compression perpendicular to grain (kg/cm²)	Green 12% MC	90.0 135	56.0 90.0	25.5 58.0	22.5 37.5	14.0 24.5
Shear parallel to grain (kg/cm²)	Green 12% MC	100 140	80 110	63.0 85.0	50.0 65.0	40.0 50.0
Specific gravity*	Green 12% MC	0.670 0.710	0.545 0.580	0.450 0.475	0.365 0.385	0.300 0.315

* Based on weight when oven-dry and volume at test
* 1 kg/cm² = 0.098 MPa

(d) South America

Five South American countries, Bolivia, Columbia, Ecuador, Peru and Venezuela under the auspices of the Andean Pact have in recent years undertaken a comprehensive testing program aimed at developing a set of grade rules and a strength grouping system applicable to the region. This was the subject of a detailed report by Centeno (1978). In this the advantages of a strength grouping system are stated as follows:

(i) it permits the introduction of a large number of new little-used species to the building industry;

(ii) it allows a more homogeneous, balanced and rational exploitation of the forest;

(iii) it allows the limitation or elimination of the vices implicit in the selective exploitation of a few precious species; and

(iv) it drastically simplifies the use and commercialization of wood as a construction material.

As a result of the above study, these five countries have agreed on a single visual grading rule for structural hardwood and a strength grouping system comprising three strength groups. The working stresses derived for each strength group were arrived at after taking cognizance of both the results available from small clear testing of 72 species and the testing of approximately 1500 beams of structural size timber representing more than 30 species.

The proposed working stresses for the three strength groups are as given in Table 10. These values are derived by taking the lowest 5th percentile value for the group. The minimum Modulus of Rupture values are then divided by 2.1 to account for accidental overload and the effect of duration of load; a further reduction of 10 per cent is applied to account for a further size effect. The Modulus of Elasticity values are the averages taken directly from the tests without further modification.

TABLE 10

PROPOSED WORKING STRESSES (kg/cm²)* (Centeno 1978)

Group	Fm Flexure	F_c Comp. Para.	F_t Tens. Para.	F_p Comp. Perp.	F_v Shear		E Modulus of Elasticity	
					Beams	Joints	$E_{0.5}$	$E_{0.05}$
A	220	170	160	60	20	25	140	110
B	170	130	120	45	16	20	120	95
C	130	100	90	30	12	15	90	70

1 kg/cm² = 0.098 MPa

For a new species to be classified under the proposed system it is recommended that at least 60 beams be tested in third point bending and that the 5th percentile MOR values (modified as above) and the mean MOE values be used to determine the correct strength group by direct comparison with the Table. A species may be allowed in a particular group when these parameters are no more than 10% lower than the values indicated.

During the course of the testing program it was observed that basic density was a good indication of strength and as a consequence basic density is now proposed as a method of positioning a species in a group on a preliminary basis. The limits selected taking a conservative approach were as given in Table 11.

TABLE 11
LIMITS FOR BASIC DENSITY (gm/cm^3)
FOR EACH STRENGTH GROUP

Group	Basic Density*
A	0.76 and above
B	0.60 - 0.75
C	0.44 - 0.59

* Basic density is the denisty of timber calculated from the green (or fully saturated) volume and the mass when oven dry.

An interesting approach taken in the development of the single visual grading rule was that the limits set on size and location of defects should permit an average mill to produce 50-60% of acceptable structural material. The remainder of the mill output would normally be suitable for non-structural applications in housing such as sheathing and joinery.

As a consequence of the acceptance of the above system, a Timber Constuction Manual has been produced and industry has expanded as is evidenced by the establishment of nine factories producing prefabricated houses in the five countries concerned and the construction of a wood/cement panel plant in Ecuador. It is noteworthy that the various governments support the rules and are incorporating them into the relevant building codes.[1]

The incentive for the Andean Pact countries to develop a stress grading and grouping system was the assistance it would provide in overcoming the serious housing shortages, the need to utilize a valuable resource and the need to create employment.

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[1] As stated during the Expert Group Meeting on Timber Stress-Grading and Strength Grouping, Vienna, 14-17 December 1981.

(e) Mexico

In Mexico (Davalos 1981) development of a simplified set of grading rules is close to being finalised. The 50 pinus species in used throughout the country have for convenience been treated as a single species group. A large in-grade testing program (5000 full-sized pieces) is in progress to determine the appropriate working stresses for the two grades of structural timber considered necessary. Up until now, North American grading rules have been used but their validity has been queried prompting the above testing progam.

The proposed grading rules have been framed so that, on average, mill output would be 30 per cent of the top grade, 40 per cent into the second grade, with the remainder going into non-structural applications. If this break-down can be reflected throughout the country there would be sufficient production to fulfil the needs of the local market.

The tentative design values based on the tests to date for the two suggested grades of pine are given in Table 12.

TABLE 12
TENTATIVE DESIGN VALUES FOR MEXICAN PINE (kg/cm²)*

Grade	Bending		Mean M of E x 10³
	Single Members	Load sharing Members	
A	140	160	115
B	80	90	90

* 1 kg/cm² = 0.098 MPa

Investigations are also under way in an attempt to obviate the need for visual grading. A TRU-grader has been purchased from South Africa and is currently being evaluated in the field.

Also being examined is the indication that for Mexican conditions the within mill variation is larger than the regional variation. If this is the case, sampling will be much simpler than previously thought and considerably less expensive. This could be of interest to the countries with a widespread pine resource.

In Mexico it is felt amongst research workers that the lack of a suitable timber grading system is the main reason why industrialised housing using timber framing has not become established. It is felt that timber frame construction could help considerably to lower housing costs below those incurred with traditional masonry materials and so make home ownership accessible to a larger proportion of the population.

(f) Other Countries

There are several other countries, particularly those with a hardwood resource, that have strength grouping systems similar to or based on the Australian system. In Canada and the United States grouping techniques are used for the comparatively small number of species used structurally, but the degree of refinement attempted appears to have little relevance to the problems of developing countries. However, in the broader sense, much of the research work emanating from both Canada and USA has important implications for other countries but with the proviso that naturally most of the work is on softwood species. Of particular interest are the developments arising as a result of in-grade testing research programs and the detailed studies being undertaken to determine the duration-of-load effect.

(g) Summary

While the strength grouping/stress grading systems described so far vary somewhat in the way they have been developed and are being utilized they still have much in common. Firstly, they all use a small number of groups to cater for a comparatively large number of individual species, secondly they each aim to estimate the influence of defects on strength and stiffness, and thirdly they are all based on visual grading. Their commonality becomes more obvious when working stresses are developed for some well known species using the different systems. Having regard to the lack of accuracy inherent in the concepts as a whole the end-results are very similar.

It is when attempts are made to bestow on any of the systems a degree of precision that is really not there, nor warranted, that apparent discrepancies arise.

5. EXTENSION OF THE GROUPING TECHNIQUE

(a) Joints

It has been found that grouping is also a very useful technique in developing the basic loads applicable to metal fasteners (Mack 1978). When revised, the Australian Timber Engineering Code will be using the following classification system based on basic and air-dry density as shown in Table 13.

TABLE 13
PROPOSED MINIMUM DENSITY FOR JOINT STRENGTH GROUPS

Green timber		Seasoned timber	
Group	Basic density (kg/m³)	Group	Air-dry density* (kg/m³)
J1	750	JD1	940
J2	600	JD2	750
J3	475	JD3	600
J4	380	JD4	475

* Density at 12% moisture content after reconditioning

An example of its application to nailed joints is given in Table 14.

(b) Poles

From Tables 4 and 5 it can be seen that if the product under consideration had only one visual grade and one moisture condition, then a much simplified new table would be possible. This is the case with poles if they are graded to the Australian Standard.

On the basis of a large pole testing program carried out by the Commonwealth Scientific and Industrial Research Organization (Boyd 1962) poles from mature trees are considered to be in a single grade, the next above the 75% grade. As poles are normally regarded as unseasoned, then Table 4 leads to Table 15.

TABLE 14
PROPOSED MINIMUM PROPERTIES OF NAILED JOINTS
LOADS ARE FOR 2.8 mm DIAMETER NAILS IN SINGLE SHEAR

Green timber				Seasoned timber*			
	Minimum value				Minimum value		
Group	Maximum load (N/nail)	Load at 0.4 mm displacement (N/nail)	Stiffness modulus	Group	Maximum load (N/nail)	Load at 0.4 mm displacement (N/nail)	Stiffness modulus
J1	2170	685	1220	JD1	1920	925	1420
J2	1710	505	895	JD2	1490	700	1110
J3	1330	365	650	JD3	1170	530	875
J4	1050	270	480	JD4	905	395	680

* Approximately 12% moisture content

TABLE 15
CORRESPONDENCE BETWEEN STRENGTH GROUP AND STRESS GRADE FOR ROUND TIMBERS GRADED TO AS 2209-1979

Strength group	Stress grade
S1	F34
S2	F27
S3	F22
S4	F17
S5	F14
S6	F11
S7	F8

NOTE: The equivalence expressed is based on the assumption that poles or logs are from mature trees.

(c) Plywood

A similar development is possible with plywood. In the Australian Standard 2269-1979 for structural plywood (Standards Association of Australia 1979c) visual grading rules for plywood veneer are specified so that their strength is roughly 60 per cent of the clear wood strength when the maximum permissible defects occur. With this prerequisite satisfied, the stess grade for a plywood may be derived from any one of the following three parameters:

(i) the strength group of the timber veneer;
(ii) the density of veneers; or
(iii) The stiffness of the plywood sheets.

Table 9 shows the relationship between these parameters and the plywood stress grade. In this table, the modulus of elasticity refers to the value of stiffness of solid wood parallel to the grain that must be used in computing the stiffness of the plywood sheet.

TABLE 16
GRADING PARAMETERS FOR PLYWOOD STRESS GRADE

Plywood stress Grade	Grading parameters*		
	Timber strength group	Modulus of elasticity of plywood sheet (MPa)	Minimum air-dry density (kg/m³)
F34	SD1	21 500	1 200
F27	SD2	18 500	1 080
F22	SD3	16 000	960
F17	SD4	14 000	840
F14	SD5	12 200	730
F11	SD6	10 500	620
F8	SD7	9 100	520
F7	SD8	7 900	420

* Only one of the three grading parameters need be used

(d) Non-Structural Properties

There are several properties that may be classed as non-structural, e.g. durability and shrinkage, but are still of critical importance to the engineer. Suggested methods for classifying these have been made by Keating (1981).

REFERENCES

American Society for Testing and Materials (1981). Standard Methods for Establishing Structual Grades and Related Allowable Properties for Visually Graded Lumber, D245. Philadelphia, USA.

Berni, C., Bolza, E., and Christensen, F.J. (1979). South American Timbers - the Characteristics, Properties and Uses of 190 Species. CSIRO Div. Bldg Res., Melbourne, Aust.

British Standards Institution (1967). Code of Practice on the Structural Use of Timber, CP112:1967. London.

Bryant, P.A.V. (1978). The stress grading of S.A. pine. HOUTIM, CSIR National Timber Research Institute, Pretoria, June.

Bolza, E., and Keating, W.G. (1972). The Properties, Uses and Characteristics of 700 African Species. CSIRO Div. Bldg Res., Melbourne, Aust.

Boyd, J.D. (1962). The Strength of Australian Pole Timbers. II. Principles in the Derivation of Design Stresses for Poles. CSIRO Div. of For. Prod., Technol. Paper No. 22. Aust.

Centeno, J.C. (1978). Andean Grading System for Structural Hardwood Timber. Working Stress-Strength Groups. Forest Products Research Society, 32nd Annual Meeting, Atlanta, Georgia, USA.

Cooper, K.L. (1953). Preferred Stress Grades for Structural Timber. Aust. J. Appl. Sci. 4:77-83.

Dávalos, R. (1981). Development of Grading Rules for Structural Mexican Pine Lumber. Paper presented at UNIDO Meeting on Timber Stress Grading and Strength Grouping, Vienna, Austria, December. UNIDO Document ID/WG359/2.

Espiloy Jr., E.B. (1978). Strength Grouping of Philippine Timbers for Utilisation of Lesser-Known Species. For. Prod. Res. Ind. Development Commission, Nat. Sci. Development Board. Tech. Note No. 187, March.

Keating, W.G. (1981). Utilization of mixed species through grouping and standards. Aust. For. 43(4):233-244.

Keating, W.G., and Bolza, E. (1982). Characteristics, properties and uses of 362 species and species groups from South-East Asia, Northern Australia and the Pacific Region. Inkata Press, Melbourne, Aust.

Kloot, N.H. (1973). The strength group and stress grade systems. CSIRO Aust. For. Prod. Newsl. No. 394. Sept.-Oct., pp.1-12.

Langlands, I., and Thomas, A.J. (1939). Handbook of Structural Timber Design. CSIRO, Aust. Div. of For. Prod. Technical Paper No. 32.

Leicester, R.H. (1979). Proof grading a practical application of reliability theory. Proc. 3rd Int. Conf. Appl. Statist. Probab. in Soil and Struct. Eng.,Sydney, Jan.-Feb., pp.263-77.

Leicester, R.H. (1981). Grouping and selection of species for structural utilization. CSIRO, Aust. Div. of Bldg Res. Tech. Paper (Second Series) No. 39.

Leicester, R.H., Breitinger, H.O., and McNamara, R. (1981). Damage due to proof loading. Proc. 20th Forest Products Research Conference. Topic E1. CSIRO Div. of Chem. Tech., South Melbourne, Aust.

Leicester, R.H., and Keating, W.G. (1982). The use of strength classifications for timber engineering standards. CSIRO Aust. Div. Bldg Res. Tech. Paper (Second Series) No.43.

Mack, J.J. (1978). The grouping of species for the design of timber joints with particular application to nailed joints. CSIRO Aust. Div. Bldg Res. Tech. Paper (2nd series) No. 31.

Mettem, C.J. (1981). The principles involved in stress grading with special reference to its application in developing countries. UNIDO. Meeting on Timber Stress Grading and Strength Grouping, Vienna, Austria. December. UNIDO Document ID/WG359/3.

Pearson, R.G. (1965). The establishment of working stresses for groups of species. CSIRO Aust., Div. For. Prod. Technol. Paper No. 35.

Pong Sono (1974). Merchantable timber of Thailand. Symposium on Research and Marketing of South Asian Timbers and Timber Products, organized by the German Foundation for International Development (Berlin) and the Government of the Republic of the Philippines, Manila and Los Banos, Philippines, 19-23 November, pp.120-65.

Standards Association of Australia (1973). Visually stress graded radiata pine for structural purposes, AS 1490-1973.

Standards Association of Australia (1974). Visually stress graded cypress pine for structural purposes, AS 1648-1974.

Standards Association of Australia (1977a). Visually stress graded hardwood for structural purposes, AS 2082-1977.

Standards Association of Australia (1977b). Visually stress graded seasoned Australian-grown softwood (conifers) for structural purposes (excluding radiata pine and cypress pine), AS 2099-1977.

Standards Association of Australia (1979a). Report on strength grouping of timbers, MP45-1979.

Standards Association of Australia (1979b). SAA Timber Framing Code, AS 1684-1979.

Standards Association of Australia (1979c). Structural Plywoood, AS 2269-1979.

Standards Association of Australia (1979d). Timber poles for overhead lines, AS 2209-1979.

THE PROPERTIES AND END USES OF A RANGE OF WOOD-BASED PANEL PRODUCTS[1]

Kevin J. Lyngcoln[2]

INTRODUCTION

The range of wood based panel products has grown dramatically during the last 20 years. There have been many new wood based panel products introduced and, through technical innovation, modifications to existing wood based panel products since the World Consultation of Wood-Based Panels held in New Delhi in 1975 (9). In 1970, there were three basic types of wood panels, plywood, particleboard and hardboard. In 1982, the basic types of wood based panels and variations within each type include oriented strandboard (OSB), medium density fibreboard (MDF), structural flakeboards and waferboards, composite panels as well as innovations within the conventional plywood, particleboard and hardboard products.

The objectives of this paper are:

1. To describe the main types of wood based panel products in terms of wood material, geometry and adhesive binder.
2. To compare the main structural properties of each product type.
3. To relate the physical and mechanical properties to the suitability of each product to perform satisfactorily in a range of end uses, mainly in construction, under the range of climatic conditions experienced in the Asia-Pacific region.

It became obvious when studying the references that to meet the above objectives it would be necessary to comment in detail on the various approaches to the structural application of wood based panels which exist currently. The approaches range from traditional prescription standards to straight performance standards and combinations of both.

It also was decided to segregate the panels into basic groups, structural and non-structural based on the long term durability of the adhesive binder. This paper will deal mainly with structural wood based panels.

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[1] This lecture was first presented in New Delhi during FAO's Technical Consultation on Wood-based Panels held 13-17 January 1983, and is reproduced with FAO's permission.

[2] Engineer, Plywood Association of Australia.

Finally, it was decided that before any of the above objectives could be dealt with some of the reasons for the development of the new wave of wood based panels, particularly those aimed at structural applications, should be considered.

Based on the literature reviewed, it must be stated at the outset that this paper does not provide detailed answers to all of the questions posed in the broad objectives. Moreover, it is expected that a great deal of additional research, in some cases of a fundamental nature, will be required to obtain accurate answers. The paper provides a review and shows additional areas that need researching if any country in the Asia-Pacific region is to optimise its production of wood based panels to suit its application requirements relative to technical ability, available capital for investment and people and forest resources.

REASONS FOR DEVELOPMENT OF ALTERNATIVE WOOD BASED PANELS

Since the mid 1970s, there have been considerable efforts made, particularly in North America, but also in Europe, to develop structural wood based panel products to replace plywood in construction applications. It is suggested (24) that the reasons for the trend can be categorised and analysed as far as the North American industry is concerned under the headings 'Technology Rated - Developments', 'Raw Material Developments', 'Market Developments' and 'Political Developments'.

Regarding technological developments, it is suggested that in North America, although the technology has existed for many years to produce high quality performance non-veneer wood based panels on a laboratory scale, it is only recently that production scale technology has been able to duplicate this acceptable quality level in commercial quantities. The recent rapid process development has occurred due to the following raw material, market and political developments.

The raw material pressures to change, result from the increased cost and lower availability of high quality peeler logs. The peeler log prices continued to inflate for the first time in history through the recent North American recession. Simultaneously there is an availability of low cost, previously ignored secondary species such as trembling aspen in Canada and spruce and light hardwoods in the upper mid-west of the USA with density and flaking characteristics which make them attractive to the new breed of structural wood based panels.

It is believed noteworthy that with the availability of lower cost plantation softwood species in Australia and New Zealand and high quality raw material availability in Malaysia and Indonesia, these trends towards the development of other structural wood panels including fibre aligned particleboard, have been minimal in those countries.

The increased peeler log price and reduced availability to the North American plywood industry has resulted in a higher priced, lower quality plywood in the marketplace. Therefore, in traditional uses such as roof and wall sheathing and subflooring, plywood no longer holds the dominant competitive edge that it has held for decades. So as not to lose these large volume markets to competitive non-wood panels, traditional plywood producing companies have developed the lower cost wood based alternative such as oriented strandboard, structural flakeboard, waferboard and COMPLY*.

Finally, in North America there has been a philosophical change in the type of standard used for wood panels. The traditional prescription approach as detailed in Product Standard PSI-74 (48) and ANSI AS 208 (26) has been supplemented by the acceptance of APA performance rate panels. Performance standards for specified applications facilitate speed acceptance in to building codes of a wide range of panel types and production techniques.

It is believed that performance standards must relate closely to end use and the climate under which the panels are to be used. The development of performance standards in North America will be discussed in detail later in this paper.

It must be stressed here that although it can be shown that the above reasons for the development of structural wood based alternatives to plywood are legitimate and logical in North America and Europe, before an Asian or Pacific country embarks on the production of similar wood panels, they would be well advised to ensure the same reasoning exists.

The new breed of wood based panels require generally a higher cost, higher technology plant and lower labour component. The resultant panel is no better structurally than plywood and so far, does not have the proven properties and range of applications both external and structural of plywood.

Discussions with machinery suppliers in Australia have revealed the following ratios of capital investment per m³ of panel produced in 24 hours and manpower (number of men) required per m³ of panel produced in 24 hours.

TABLE 1
CAPITAL INVESTMENT AND MANPOWER REQUIREMENTS
(m³/24 HOURS)

Product	Capacity output 24 hours (m³)	Capital investment per m³ per 24 hours (US$)	Manpower/m³ per 24 hours
Plywood	120	33 000	0.42
Particleboard	100	95 000	-
	300	53 000	0.20
Medium density fibreboard	300	64 000	0.20
Oriented strandboard	300	64 000	0.20
Waferboard	300	72 000	0.20

TYPES OF WOOD BASED PANELS

The long term durability of a wood based panel is closely related to the durability of the bond, provided the wood component is protected from biological attacks either by the lack of a hazard due to the end use, e.g. dry interior, or by preservative treatment if a hazard exists. It has been shown (13) that the bond durability is related closely to the chemical type of adhesive and the conditions of exposure. It is therefore believed reasonable to classify the range of wood based panels into non-structural, semi-structural and structural, based on the long term durability of the adhesive type and thus the ability to continue to carry sustained load under conditions of semi-exposure or cyclic humidity changes as experienced in the Asia-Pacific region.

It is well recognised that a short term physical test for bond durability related to long term exposure such as the 72 hour boil A bond test in AS 2098 (38) or the vacuum pressure soak as detailed in PSI-74 (48) need also to be carried out to verify the performance of a particular resin formulation within the broad chemical grouping. It is also true that before a panel can be classified as structural, physical and mechanical properties must be established. The establishment of these properties will be discussed later in this paper.

It is believed reasonable, however, that bond durability under the conditions of use is a prime consideration. Table 2 groups the wood based panels into the above-mentioned classifications under the climatic conditions prevalent throughout the Asia-Pacific region. The basis for the durability and hence structural rating, is the research work on adhesives durability carried out at the Forest Products Laboratory, Princes Risborough (13).

TABLE 2
CLASSIFICATION OF WOOD BASED PANELS[1/]

Panel type	Adhesive/binder(1)	Classification
Plywood	Phenolic - natural or synthetic (PF)	Structural
	Melamine urea formaldehyde (MUF)	Semi-structural
	Urea formaldehyde (UF)	Non-structural
Particleboard, random orientated, layered or homogenous, including 'flakeboard'	PF	Structural
	MUF	Semi-structural
	UF	Non-structural
Hardboard (fibreboard)	Naturally occuring polyphenolic chemicals, e.g. lignin	Structural
Medium density fibreboard (MDF)	PF	Structural
	UF	Non-structural
Oriented strandboard (OSB)	PF	Structural
Waferboard	PF	Structural
COMPLY*	PF	Structural
	Isocyanate(2)	-

(1) The adhesive/binder types relative to board type were established from a great number of the cited references

(2) Lack of information on long term durability of isocyanate binders (17) prevents the classification of this panel at this time

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[1/] For the further treatment of the suitability of wood-based panels for developing countries, see UNIDO document ID/WG.335/16 "Guidelines for the selection of options in establishing wood-based panel industries in developing countries" by a panel of Chinese and UNIDO-appointed consultants at the Seminar on Wood-based Panels and Furniture Industries, Beijing, China, 23 March - 7 April 1981.

The various panels types are discussed below:

(a) Plywood

Plywood is defined (41) as 'a panel consisting of an assembly of veneers bonded together with the grain in most of the alternative plies at right angles'. Each ply consists of a rotary or sliced cut veneer. Plywood is manufactured from indigenous or plantation pines, e.g. shorea spp., or eucalypts, e.g. karri, and combinations of species. Although a typical density range is 500 to 750 kg/m³, plywoods are manufactured with densities ranging from 400 to 1200 kg/m³.

Individual veneer thicknesses for rotary peeled veneer range from 0.8 to 4.5 mm. It is more usual that in practice veneer thicknesses range from 1.0 to 3.2 mm. The panels are hot pressed in flat presses and, depending on end use, are bonded with phenolic, melamine, ureas or copolymers.

(b) Particleboard (including Flakeboard)

The variables in particleboard are raw material resource, chip geometry and method of chipping, particle alignment, homogenity through the thickness, adhesive, density and thickness. With this number of variables, it is impossible to describe particleboard in anything but broad terms. Particleboard can be platen pressed or extruded. The production method also affects properties. Each variable is dealt with below in general terms (10).

Originally particleboard utilised wood waste such as planer shavings, chipped lumber and hammer milled veneer which resulted from other wood processing or fabrication lines. Although particleboard still has a waste component, much is now produced from forest thinnings in the form of round wood and secondary species which have been unsuitable for processing into timber or plywood (9,10).

Chip geometry plays an important part in the basic physical and mechanical properties of particleboard.

Geimer et al. (10) explain for example, 'dimensional stability was closely related to flake or particle geometry and quality with best thickness stability associated with small particles and best linear stability associated with large flakes'.

Particles range in size from 50 mm x 50 mm x 0.5 mm in the case of ring flaked particles from 50 mm chips to fine particles 15 mm x 1 mm x 0.1 mm resulting from planer shavings hammer milled without screen.

Particleboard may have the particles randomly orientated which provides about equal properties in both directions, or have the particles aligned to give improved stability and strength in the direction of fibre alignment. The boards may be homogeneous through the thickness, or multi-layered, usually three, to give improved surface characteristics and mechanical properties (10).

Particleboards are usually manufactured from 3 mm to 25 mm thicknesses using the platen pressed mat formed process, from 25 mm to 75 mm using the extrusion process and from 2 mm to 9 mm thick using the continuous process.

The density range for what are considered medium density particleboards is 650 to 810 kg/m³. Density above 810 kg/m³ is considered high density particleboard, below 650 kg/m³ is considered low density (26).

Adhesive binders are similar to plywood, i.e. phenolic, melamine/urea, copolymers and urea formaldehydes, depending on the intended end use of the board.

(c) Hardboard (Fibreboard)

Hardboard is a panel manufactured from inter-felted ligno-cellulosic fibres which are consolidated under heat and pressure in a hot press to a density of 500 kg/m³ or greater. Other materials may be added to improve certain properties such as stiffness, hardness, resistance to abrasion and moisture, as well as to increase strength, durability and utility (49).

Standard hardboard is usually between 800 and 1000 kg/m³ density while oil tempered hardboard is slightly higher, between 950 and 1200 kg/m³. Both panel types are usually manufactured in thicknesses 3 to 13 mm (9).

Panels fabricated by the wet form method, which is used in Australia, are smooth on one side only and have a screened back. No binders to improve adhesive properties are used in this process.

(d) Medium Density Fibreboard (MDF)

Medium density fibreboard is a panel product manufactured from ligno-cellulosic fibres combined with synthetic resin or other suitable binders. The panels are manufactured to a density of 500 to 800 kg/m³ by the application of heat and pressure by a process in which the inter fibre bond is substantially created by the added binder. Other materials have been added during manufacturing to improve certain properties (27). Using the drying process, which is normal in Australia and New Zealand, densities are between 600 and 800 kg/m³ and thicknesses 3 to 50 mm. The adhesive binder is usually urea formaldehyde which renders the bond unsuitable for structural applications. Discussions with researchers suggest that by adding phenolic or tannin based binders a durable structural panel can be achieved (5).

(e) Oriented Strandboard (OSB)

OSB is a specialised particleboard product. It is manufactured from aligned strands of wood which possess much greater length than width. Typical dimensions for an individual strand are 40 mm in length by 6 mm in width and 0.4 mm thick. OSB is usually a three-layer board in which the top and bottom layers are oriented in the direction of the panel length while the inside layer is aligned in the direction of the panel width. In essence the OSB configuration mimics plywood's construction.

Usually the two outside layers each contain 25 per cent of the wood particle and the centre layer 50 per cent.

OSBs use liquid phenolic resin at about 5 to 6 per cent on a dry solids weight for weight basis. An average density value for OSB is 680 kg/m³. The usually available thicknesses for OSB range from 8 to 20 mm (4,24).

(f) Waferboard

Waferboard is manufactured generally, as previously reported, from low cost secondary species available throughout North America. The most common species is trembling aspen, however spruce and some lower density hardwoods are also used. The ability of the species to 'flake' properly, as is the case with the above-mentioned species, is a prerequisite of suitability for waferboard production. Radiata pine possesses fair flaking characteristics which may be significant as far as the Asia-Pacific region is concerned.

The particle size used in waferboard production is generally large relative to other particleboard types. Particles 40 to 75 mm square by 0.775 to 1 mm thick are standard. The boards can have the wafers randomly orientated or aligned depending on the properties of the board that are required. Normal waferboard has a thickness range from 12 to 25 mm and average density of 660 kg/m³. Waferboards are usually manufactured commercially using powdered phenolic resins (23,24,46).

(g) COMPLY®

COMPLY® refers to a range of panels consisting of basically a phenolic bonded particleboard overlaid on both sides with veneer. The core may be randomly orientated particleboard, OSB or waferboard. The fibres in the core may be aligned at right angles to the direction of the face veneer to give improved dimensional stability in the cross direction.

The panels can be made in a two pass operation by firstly manufacturing the core and then gluing the overlay veneers in a second operation. One US manufacturer is using a one pass operation by pressing the veneers and particle mat in one pressing. This company is using isocyanate adhesives. COMPLY® has a density range which depends on veneer species and core type. From the literature (24) 630 kg/m³ appears to be an average value for material produced from US southern pine. Presently the thickness range in COMPLY® appears to be between 12 and 25 mm.

STANDARDS

Before comparing the physical and mechanical properties and applications of the wood based panels described in the preceding section, it is necessary to comment on the types of standards under which wood based panels have been manufactured and marketed, and any current changes in the philosophical basis for these standards. The reasons for change are also an important issue which is tied to structural properties and application.

Until relatively recently all wood based panels have been manufactured and marketed universally to prescription type standards. Prescription standards are founded on the philosophical base that if the ingredients and manufacturing processes and methods are rigidly enforced then the product will have known performance. The US and Australian standards for plywood (39,48) and particleboard (27,43) are representative of this type of standard.

Prescription or manufacturing standards have been used reasonably successfully for structural timber and plywood in countries where a small number of species are used, e.g. USA, Canada and Finland. In the Asia-Pacific region, due to the multitude of available species (e.g. 90 species are available to the Australian Plywood industry), prescription standards are inherently conservative and restrictive. The conservative and restrictive nature of prescription standards in countries with a larger number of available species is due to the necessity to group species (16) so that unit stresses need only to be applied to a few groups rather than a multitude of species, e.g. AS 2269 (39) has unit stresses for eight stress grades rather than for the 90 individual available species. The grouping method leads to a lowest commom denominator approach based on the properties of the weakest species in the group and is restrictive because every new available species must be tested so that it can be assigned to a group.

The Australian Standard AS 2269 has endeavoured to overcome the restrictive and conservative nature of the prescription approach by including a performance section. The performance test is used to establish the stiffness (Modulus of Elasticity E) of any plywood panel manufactured from any species or group of species.

Then, because of the well established relationship between (E) and the other basic strength properties, a grade mark can be applied. The grade mark has associated unit stresses prescribed in the standard. A production machine 'stress grader' has been developed to carry out the stiffness sorting. Although the machine grading has reduced the conservative and restrictive nature of the prescription approach to applying unit stresses to Australian plywood, it must not be overlooked that its use is based on the relationship between (E) and the other strength properties which was established by a great number of tests.

Prescription standards were a reasonable basis for the standardisation of reconstituted wood products while the panels were used for non-structural applications such as door skins or furniture where the major consideration was finish, not structural performance. For the reasons substantiated previously in this paper, particleboard, OSB, waferboard, MDF and COMPLY* now are being aimed at structural markets as plywood alternatives particularly in light framed construction. To facilitate the acceptance of these products quickly into building regulations and so as not to be over conservative or restrictive in their application, there is a school of thought in North America, Australia and New Zealand (1,25,28,30,37) that fosters performance standards against the traditional prescription approach.

A performance standard is defined by the American Plywood Association (1) as follows:

'A performance standard is oriented towards the end use of the product and does not prescribe by what means the product will be manufactured. The overall objective is to assure, for a particular end use, that the product will satisfy the requirements of the application for which it is intended. To do this the performance criteria must address the critical attributes of the product that will assure successful performance in the market place. This necessitated the development of new and innovative test methods, each linked to field requirements. Therefore, under performance testing the qualification process correlates the attributes of the product to the market place'.

To qualify under performance testing, a product is evaluated to ascertain its compliance with a series of performance requirements relative to a specific end use. The product must demonstrate for example that structural adequacy, stability and durability necessary to perform in service. Simultaneously, properties such as density, internal bond, and strength retention are also measured. A product specification is written based on these latter properties, that can be easily measured on a sustained quality control basis. The minimum values are set and a mill must, through internal quality control, make sure the minimum values set in the product specification are achieved. The mill is independently audited then on a regular basis to ensure firstly, the mill's quality control is operational and secondly, that a sample of the product when tested still meets the desired performance criteria.

The APA already has performance standards for roof and wall sheathing and single layer flooring. The Australian particleboard industry has a performance standard AS 1860 forsingle layer flooring (37).

It must be stressed that performance standards must be tied closely to specific end use and do not allow for the application of unit stresses.

There is no doubt, however, that it is possible to assign unit stresses to reconstituted wood based panels using the prescription method or machine grading methods described above (31). The problems confronting the end use of this traditional approach are the large range of products available, the volume of testing that needs to be done on an industry basis, large coefficients of variation (C_v) across an industry leading to low unit working stresses, the time required to carry out testing and statistically evaluate the results and the continuing introduction of new products.

If reconstituted wood products are to be truly structural panels, unit stresses must be applied. It is believed by some (24) that one solution is to set up standard test methods and statistical bases for applying unit stresses which can then be applied to specific products by individual companies. This approach seems sensible. The following quote really sums up the North American scene and the structural use of reconstituted panels generally (24).

'It is unclear whether dissemination of engineering design values for the new panels will be handled more expeditiously via the peeformance or the prescription route. The former will require each manufacturer to state and support design values for his product, while the latter could be managed so that single values could be used to represent the products of all manufacturers producing like products. Considerble work has been done towards the derivation of allowable stress values for panel products in support of the latter approach'.

STRUCTURAL PROPERTIES

It became apparent from studying the references that unit stress values, sometimes termed basic working stresses were available for plywood only. For comparative purposes therefore it became necessary to compare the structural properties of the panel types listed in Table 2 using mean values for Modulus of Elasticity and Modulus of Rigidity and average ultimate values for the remaining structural characteristics.

Table 3 gives an average or range of average values for Modulus of Rupture, Modulus of Elasticity, average ultimate tension, compression and panel shear strengths and the Shear Modulus (Modulus of Rigidity). The values given in Table 3 were extracted from a great number of the sources listed in the references. Where no information could be found, a dash is shown in the Table. The values in the table are ultimate average mean values and are for comparative purposes only. The values must not be used for engineering design purposes.

It is believed noteworthy that internal bond strengths were not included as the literature showed the values published by FAO (9) needed no modification.

The values in Table 3 for fibre aligned boards or plywood are parallel to the grain or direction of fibre alignment. The values for other randomly orientated particleboards or fibreboards are parallel to the length of the panel.

TABLE 3 COMPARISON OF STRUCTURAL PROPERTIES

Panel type	Modulus of rupture (MPa)	Modulus of elasticity (MPa)	Ave. ultimate tensile strength (MPa)	Ave. ultimate compressive strength (MPa)	Ave. ultimate shear strength (MPa) (panel shear)	Modulus of rigidity (MPa)
Plywood						
Australian low density (F_8)(1)	28.4	9 100	22.7	21.5	5.3	525
Australian medium density (F_{14})	46.2	12 500	36.3	34.6	6.8	625
Australian high density (F_{22})	72.5	16 000	56.0	54.4	7.6	800
USA West coast	29.0	9 600	-	41.0	6.9	590
USA Southern coast	27.5	9 600	-	41.0	6.9	-
Particleboard - random orientated						
USA high density	-	-	14-34	17-36	-	-
USA medium density(2)	17-22	2750-4000	8-16	9-21	6-10	1200-1600
USA low density	-	-	6-8	3-10	-	-
Europe medium density	14.6-28.5	2500-4000	7.3-12.7	10.2-16.5	5.8-7.6	970-1230
Aligned three layer	25.4-55.8	9500-11000	11.7-21.3	15.2-26.0	6.5-11.2	-
Hardboard						
Tempered	45-75	5000-6500	26-54	26-41	16-20	-
Standard	30-65	3000-5000	21-41	12-41	12-16	-
Medium density fibreboard	32-34	3300-3650	17.2-21.4	20.0-24.8	-	-
Oriented strandboard (OSB)	34.4	9 600	(3)	(3)	(3)	-
Waferboard	17.2	3400-4800	-	11.0-12.4	8.3-9.6	1 500
COMPLY*	31.0	9 600	-	-	-	-

(1) The (F) refers to stress grade as detailed in AS 2269
(2) Medium density particleboard in the density range 650-800 kg/m^3 is the most widely used in the USA
(3) It is believed these properties would be similar to the fibre aligned three layer particleboard

A ratio that is significant from the viewpoint of transport costs and ease of handling on site, important in residential building or formwork applications, is the structural properties of the materials per unit weight. As flexural properties are usually important criteria in wood based panel utilisation, Table 4 details the strength and stiffness to weight ratios of the range of panels under consideration.

Strength/Weight Ratio = (Modulus of Rupture)/(Average Density)
Stiffness/Weight Ratio = (Modulus of Elasticity)/(Average Density)

TABLE 4
STRENGTH AND STIFFNESS TO WEIGHT RATIOS

Product	Average density (kg/m^3)	Strength/ weight ratio	Stiffness weight ratio
Plywood low density	520	0.055	17.5
Plywood medium density	730	0.063	17.1
Plywood high density	960	0.075	16.7
Particleboard			
Random orientated medium	720	0.028	4.7
Fibre aligned medium	720	0.055	14.2
Hardboard			
Tempered	1 075	0.047	5.3
Standard	900	0.052	4.4
Medium density fibreboard	700	0.047	4.8
Oriented strandboard	660	0.051	14.5
Waferboard	670	0.026	6.1
COMPLY*	620	0.050	15.4

The above ratios show that on the basis of structural properties per unit weight, plywood, aligned particleboard, OSB and COMPLY* have similar characteristics. Waferboard has poor ratios in both cases which probably indicates that the manufacturing installations in the USA and Canada are close to high population densities rather than near to the forest resource.

Another major consideration in the structural utilisation of any timber or wood based panel is creep. Creep is the ability of a material to resist increases in deflection additional to initial deflection whilst under sustained dead load. Steel does not creep whereas all timber and wood based materials do. The creep characteristics of timber and plywood are well established, in fact a multiplying factor of 2 is applied to the short term deflection for dry timber and plywood products carrying sustained load (36).

Creep in wood is affected by conditions of exposure and cyclic humidity. It must be allowed for in any application of wood products involving sustained loads. Domestic floors which carry heavy refrigerators or roof sheathing carrying hot water services are examples of applications requiring consideration of creep.

From the literature received, it appeared that there was not a great amount of information available on the creep characteristics of reconstituted wood products. One reference (22) does, however, compare the creep charateristics of three layer flakeboards with random core and face flakes aligned in the direction of the panel with Douglas fir and southern pine plywood.

Because the flakeboards were manufactured at Forest Products Laboratory, Madison, it is assumed they were bonded with phenolic adhesive with 5 to 6 per cent phenolic used on a dry weight basis. Creep specimens were loaded for 90 days under both constant humidity 65% RH at 32°C and cyclic humidity 25% to 85% RH at 26°C. The results are given in Table 5.

TABLE 5
CREEP BEHAVIOUR OF PANEL MATERIALS AFTER 90 DAYS LOADING

Panel type	Density (kg/m^3)	Creep deflection (additional per cent of initial deflection)	
		Constant humidity	Cyclic humidity
Aspen	745	62.2	228.6
Red oak	745	68.9	275.6
Red oak	795	61.5	248.2
Douglas fir plywood	-	43.8	191.2
Southern pine plywood	-	43.6	145.3

The table shows that although plywood has superior creep properties to the particleboard in question, it is possible to evaluate the creep characteristics of reconstituted wood products by comparing them to a material such as plywood with well defined creep characteristics.

PHYSICAL PROPERTIES

The dimensional stability of wood based panels under changes of moisture content, sometimes referred to as hygroscopic movement, is an important physical property because of its relevance to almost every application. The linear expansion (LE) and thickness swelling (TS) of each product should be known so that correct installation methods can be devised which allow for hygroscopic movement.

A good basis for comparison and design purposes is per cent movement per per cent change in panel moisture content. Values for a range of wood panels are given in Table 6.

TABLE 6
HYGROSCOPIC MOVEMENT % PER % MOISTURE CONTENT CHANGE

Product	Movement % per % MC change		
	Length	Width	Thickness
Softwood plywood 3 ply(1)	0.014	0.020	0.29
Softwood plywood 9 ply	0.017	0.016	0.27
Mixed hardwood plywood 3 ply	0.016	0.016	0.29
Mixed hardwood plywood 9 ply	0.024	0.022	0.29
Australian particleboard(2)	0.039-0.055	0.039-0.055	0.38-0.61
US particleboard (random orientated)(3)	0.046	-	0.28-0.75
US particleboard (aligned fibres)(3)	0.015	0.056	0.80
Oriented strandboard(4)	0.015	-	0.80
Waferboard(4)	0.022	-	1.60
COMPLY®(4)	0.022	-	0.50

(1) Unpublished data, CSIRO Div. of Bldg Res. reference (6)
(2) Random three layered particleboard, 650 kg/m³ average of phenolic and urea bonded
(3) References (7), (20) and (24)
(4) Reference (20)

Comparative values for hardboard and medium density fibreboard on a % movement per per cent change of moisture content could not be found in the literature. Movement between 30 and 90% relative humidity is given in the Proceedings of the World Consultation on Wood Based Panels (9). In many applications in which wood based panels are used such as wall and roof sheathing and flooring, the thermal conductivity (k) of the panel is an important product property. As heating and cooling costs become greater due to increased energy costs, construction techniques which utilise products that provide both structural performance and insulation must become more attractive. Wood based panels can provide a combination of these characteristics.

Thermal conductivities for a range of wood based panels are given in Table 7 (47).

TABLE 7
THERMAL CONDUCTIVITIES OF WOOD BASED PANELS

Product	Thermal conductivities (k) W/m²/°C per 25 mm thickness
Plywood softwood	4.5
Plywood hardwood	6.4
Particleboard low density	3.1
Particleboard medium density	5.3
Particleboard high density	6.7
Hardboard standard	4.7
Hardboard tempered	5.7
Medium density fibreboard	4.1

It is believed waferboard, OSB and COMPLY® would have similar characteristics to those of medium density particleboard given in Table 7. For purposes of comparison, asbestos cement sheeting, a common building panel, has a thermal conductivity of 22 W/m²/°C per 25 mm thickness.

END USES

It is intended finally to examine the suitability of each product for a range of end uses and climate conditions experienced in the Asia-Pacific region. To negotiate this exercise, climate conditions familiar to the writer have been selected (50). It is believed, however, that this range covers most Asian and Pacific climates. The information given in this section is based on the references already cited and the writer's experiences in developing markets for structural plywood (33, 35) particularly in applications in low rise domestic dwellings up to two storeys high. Table 8 details the four climatic conditions used in the exercise.

The writer's opinions of the suitability of each wood based panel product to perform under the range of climatic conditions given in Table 8 for applications in residential buidings are given in Table 9.

TABLE 8
CLIMATIC CONDITIONS PREVALENT IN ASIA-PACIFIC REGION

Type	Australian example	Humidity	Ave. temp. summer (°C)	Ave. temp. winter (°C)	Average annual rainfall (mm)
Tropical (cyclonic)	Darwin Cairns	High all year	35	25	864 in 3 mos.
Sub-tropical (cyclonic)	Brisbane	High summer	25	16	1 132
Temperate (warm)	Sydney	High summer	26	8	1 200
Temperate (cool)	Melbourne	High winter	26	6	1 200

The reasons for excluding products from an application on limiting the use to particular climatic conditions are as follows:

(a) Internal Fitments, Wall Panelling, Door Skins, Furniture

Urea formaldehyde bonded panels were excluded in the severe tropical environment because of lack of bond durability under prolonged humid conditions (13, 40). Particleboards with random fibre orientation were excluded under tropical conditions due to poor linear expansion and thickness swelling characteristics (6, 7, 20, 24).

TABLE 9
SUITABILITY OF WOOD BASED PANELS FOR END USES IN RESIDENTIAL BUILDINGS UNDER THE FOUR CLIMATIC CONDITIONS DETAILED IN TABLE 8

Product	Adhesive/binder	Application					
		Internal fitments furniture door skins, wall panelling	Flooring	Flooring (wet areas)	Wall sheathing	Webbed beams	Cladding
Plywood	Urea formaldehyde	2,3,4	N/S(1)	N/S	N/S	N/S	N/S
	Melamine urea formaldehyde	1,2,3,4	3,4	N/S	N/S	N/S	N/S
	Phenol formaldehyde	1,2,3,4	1,2,3,4	1,2,,4	1,2,3,4	1,2,3,4	1,2,3,4
Particleboard	Urea formaldehyde	2,3,4	N/S	N/S	N/S	N/S	N/S
	Melamine urea formaldehyde	2,3,4	3,4	N/S	N/S	N/S	N/S
	Phenol formaldehyde	2,3,4	2,3,4	N/S	2,3,4	N/S	3,4
Hardboard	Standard	2,3,4	N/A(2)	N/A	3,4	N/S	N/S
	Tempered	1,2,3,4	N/A	N/A	1,2,3,4	N/S	1,2,3,4
M.D.F.	Urea formaldehyde	2,3,4	N/S	N/S	N/S	N/S	N/S
	Phenol formaldehyde	2,3,4	3,4	N/S	3,4	N/S	N/S
OSB	Phenol formaldehyde	1,2,3,4	2,3,4	2,3,4	2,3,4	N/S	N/S
Waferboard	Phenol formaldehyde	1,2,3,4	2,3,4	2,3,4	2,3,4	N/S	N/S
COMPLY®	Phenol formaldehyde	1,2,3,4	2,3,4	2,3,4	2,3,4	N/S	N/S

(1) N/S = not suitable
(2) N/A = not applicable

(b) Flooring

Urea bonded products were excluded from this structural application due to lack of bond durability. Melamine fortified urea formaldehyde bonded boards were excluded from tropical and sub-tropical climates due to lack of long term durability under damp or humid conditions. Phenolic bonded particleboards were restricted from use in tropical areas due to lack of panel stability under moisture content changes. OSB, waferboard and COMPLY* were restricted from use in severe tropical climates because it is believed that the present APA performance standards (1) are not sufficiently severe to relate to this environmennt. Subsequent testing may prove the use of these three products under tropical conditions.

(c) Flooring (Wet Areas)

Wet areas in a building are described as bathrooms and laundries. Because of the likelihood of continued wetting, boards bonded with other than phenolic adhesives were excluded regardless of climate on the basis of lack of bond durability under wet conditions. Under humid conditions sub-floor ventilation must be increased to allow panels to dry out. Alternatively, plywoods or other wood based panels used in wet areas should be treated against fungal attack. Phenolic particleboard, OSB, waferboard and COMPLY* were restricted for the same reasons as given for normal flooring.

(d) Wall Sheathing (Bracing)

Because wall sheathing is used to brace the house frame against horizontal forces set up by cyclonic winds or earthquakes, long term structural integrity is essential. Panels with other than phenolic bonds were therefore excluded from this application regardless of climatic conditions. Standard hardboard was excluded from use in tropical and sub-tropical climates due to moisture uptake and subsequent lack of panel durability and dimensional stability (4).

Medium density phenolic bonded fibreboard was limited for the same reason. OSB, waferboard and COMPLY* were excluded from use in tropical environments for the same reasons given for excluding them from the flooring application.

(e) Webbed Beams

Plywood was the only material included for webbed beams because it is the only material for which unit stresses could be found (39). To design an engineered fabricated product such as a web beam, unit stresses are essential.

(f) Cladding

The cladding application has been restricted to phenolic bonded plywood and tempered hardboard due to panel durability. In both cases the panels would need to be treated against fungal attack with higher preservative loadings and better penetration patterns being required as the climatic conditions become more severe. It is recommended that plywood be treated with rot preventatives regardless of climatic conditions. The treatment is essential in sub-tropical and tropical environments. Plywood cladding in Northern Australia is also usually treated against termites (42). The writer observed the successful use of particleboard cladding in temperate climates in South Africa and under test at New South Wales Forestry Commission Division of Wood Technology, Sydney.

It is believed extremely significant that although structural wood based panels and their application in residential building result mainly from applied research efforts in developed countries such as the United Kingdom, United States, Canada and more recently Australia, there are examples of situations where the technology has been transferred to developing countries with a great deal of success.

An excellent example of this type of technology transfer occurred after Cyclone Melli caused extensive property damage and life loss in Fiji in 1979. With peak wind gusts to 200+ km/hr, damage to traditional housing in Fiji was extensive. In an effort to minimise future damage to housing, the Fiji Housing Authority in close cooperation with a local timber and plywood manufacturer developed a low cost, cyclone resistant dwelling along traditional lines using locally made plywood and timber. The structural design aspects were based on specifications for cyclone housing in Australia prepared by the Plywood Association of Australia (33).

The low cost dwellings comprised a timber sub-floor, wall frame and roof frame. Phenolic bonded structural plywood was used as flooring. The walls were clad with 7 mm thick copper chrome arsenate treated structural plywood. The plywood provided both wind bracing and hold down resistance as well as fulfilling its traditional role of cladding to keep the weather out. Plywood webbed, timber flanged box and channel beams were used over window and door openings to resist wind uplift and treated plywood storm shutters were used over windows to protect occupants from flying debris.

In March 1980 the cost, including assembly for the 8 m x 5 m two-roomed dwelling was approximately US$3100. According to the Fiji Times, 14 March 1980, the dwelling contained 98.5% of Fijian made material and each dwelling utilised 572 man hours to produce the timber and plywood and for erection. Thus the dwelling under discussion used developed country technology in a developing country to provide structurally improved low cost dwellings which utilised a very high percentage of local material and labour. Similar developments have occurred in other Pacific countries such as Tonga after Cyclone Isaac.

A major end use for plywood in Australia is in concrete formwork. Thirty per cent of Australian plywood production is used for this application. The design of plywood formwork is based presently on the availability of unit stresses and standardised section properties (34). It would be possible to devise performance tests for this application and this may allow other wood based panels, particularly OSB, waferboard and COMPLY* to be used as plywood alternatives. The APA was considering the development of such a performance standard for formwork in 1980 (18). The use of wood panels for formwork would be independent of climatic conditions.

Finally there is a wide range of engineered applications in which plywood is used already (32) such as portal frames, containers, industrial beams, pallets, crates and industrial flooring which may in the long term be open to the other structural wood based panels described in this paper. These applications will only become available if unit stresses are applied to the products, or performance standards, tied to the specific end use and climatic conditions under which the panels are to be used, are developed.

CONCLUSIONS

1. The continued development of new wood based panel products will be aimed at structural applications.

2. The new types of products such as OSB, waferboard and COMPLY® have been developed as plywood substitutes to utilise hitherto non-used and therefore cheaper wood resources, e.g. aspen in Canada and spruce and low density hardwoods in USA.

3. The technology exists to produce panels with similar characteristics to plywood, however, plant costs are higher than for plywood production and manpower requirements are lower. These are believed primary considerations in Asia-Pacific decision making.

4. Types of panels produced in a country should depend on the cost and availability of raw materials, technology available to the market, climatic conditions and availability of building codes.

5. Future structural applications of wood based panels require either the development of unit stresses or development of performance standards that relate specifically to one application which should be related also to the climate under which the panel is to be used or both.

6. Research is required into the durability of new adhesives such as isocyanates and the performance of new formulations of existing adhesives such as phenolic, tannins and copolymers.

7. Medium density fibreboards will be used to an even greater extent in furniture production.

This paper has given an updated picture of the properties and applications of a wide range of panel products and has sought to give a guide to their application relative to the types of climatic conditions experienced in the Asia-Pacific region. The end use was biased towards use in residential buildings because it is believed that this is where the potential lies in the region to best utilise the forest resource to the advantage of the majority of people. In the final analysis the paper may help to provide a basis for decision making relative to the best utilisation of the different types of forest resource in the area keeping in mind the available standard of technology and the requirements of manpower utilisation.

REFERENCES

1. American Plywood Association (1980). Performance standards for policies for APA structural use panels. APA, August.

2. Baker, R.J., and Gillespie, R.H. (1978). Accelerated aging of phen-olic bonded flakeboards. Structural flakeboard from forest residues. Forest Service General Report WO-5, Forest Service, Washington, DC, USA.

3. Bodig, J., and Jayne, B. (1982). Mechanics of wood and wood composites. Van Nostrand Reinhold, New York.

4. Brown, J.H., and Bean, S.C. (1974). Acceptance of strandwood sheathing for construction uses. Proceedings of the 8th Particleboard Symposium, Washington State University.

5. Commonwealth Scientific and Industrial Research Organisation, CSIRO (1982). Disucssion with Officcers of the Adhesives Section.

6. Commonwealth Scientific and Industrial Research Organisation, CSIRO (1982). Unpublished Data, Division of Building Research, Melbourne.

7. Countryman, D.R. (1979). APA composite panel research - an update. Proceedings of the 12th Particleboard Symposium, Washington State University.

8. European Federation of Association of Particleboard Manufacturers (1979). The preliminary publication of FESYP investigations into elastomechanical properties of particleboards of importance in construction. February.

9. Food and Agriculture Organisation of the United Nations (1975). Proceedings of the World Consultation on Wood Based Panels, New Delhi, India.

10. Geimer, R.L., Lehmann, W.F., and McNatt, J.D. (1974). Engineering properties of structural particleboards from forest residues. Proceedings of the 8th Particleboard Symposium, Washington State University.

11. Hoyle, R.J. Jr (1974). Factors required for establishing structural particleboard design properties. Proceedings of the 7th Particleboard Symposium, Washington State University.

12. Hoyle, R.J. Jr (1974). Property requirements of strutural roof and floor sheathing. Proceedings of the 8th Particleboard Symposium, Washington State University.

13. Knight, R.A.G. The efficiency of adhesives for wood. Forest Products Research Bulletin 38, Fourth Edition.

14. Lehmann, W.F. (1974). Properties of structural particleboards. Forest Products Jurnal, Vol.24, No.1, January.

15. Lehmann, W.F. (1977). Durability of composition board panels. Proceedings of the 11th Particleboard Symposium, Washington State University.

16. Leicester, R.H. (1981). Grouping and selection of species for structural utilisation. CSIRO Division of Buillding Research Technical Paper (Second Series) No. 39, Australia.

17. Lyngcoln, K.J. (1980). Exterior adhesives for plywood. Report to the J.W. Gottstein Trust on Overseas Study Tour.

18. Lyngcoln, K.J. (1980). Report to Plywood Association of Australia on overseas study tour.

19. McNatt, J.D. (1973). Basic engineering properties of particleboard. Proceedings of the 7th Particleboard Symposium, Washington State University.

20. McNatt, J.D. (1974). Properties of particleboards at various humidity conditions. USDA Forest Service Research Paper, FPL 225.

21. McNatt, J.D. (1981). Update on structural flakeboard research at the US Forest Products Laboratory. Canadian Waferboard Symposium, Forintek Canada Corporation.

22. McNatt, J.D., and Hunt, M.O. (1982). Creep of thick structural flakeboards in constant and cyclic humidity. Forest Products Journal, Vol.32, No.5, May.

23. Maloney, T.M. (1981). The overlapping roles of wood based panels. 7th Asian Plywood Manufacturers Conference, October.

24. Montrey, H.M. (1982). Engineering structural panels for light framed construction. Weyerhauser Company, Tacoma, Washington, USA.

25. Morschauser, C.R. (1980). A traditional approach to acceptance of building panels. Proceedings of the 14th Particleboard Symposium, Washington State University.

26. National Particleboard Association (1979). American National Standard for Mat Formed Particleboard. ANSI A208.1, Washington, DC.

27. National Particleboard Association. (2980). MDF for interior use only. ANSI A208.2.

28. O'Halloran, M.R. (1980). The performance approach to the acceptance of building products. Proceedings of the 4th Particleboard Symposium, Washington State University.

29. O'Halloran, M.R. (1981). Performance standards - an alternative to manufacturing standards. 7th Asian Plywood Manufacturers Conference, October.

30. O'Halloran, M.R., and Erb, C.M.Jr (1981). Performance based testing for durability. Proceedings of the 15th Particleboard Symposium, Washington State University.

31. Pearson, R.G. (1977). An interim industry standard for deriving allowble unit values for structural particleboard in bending. Proceedings of the 11th Particleboard Symposium, Washington State University.

32. Plywood Association of Ausstralia (Revised 1979). Technical Manual.

33. Plywood Association of Australia (1980). Plywood in residential construction.

34. Plywood Association of Australia (1982). Plywood in concrete formwork. Edited by K.J. Lyngcoln.

35. Plywood Association of Australia (1982). Structural plywood wall bracing - design manual.

36. Standards Association of Australia (1975). SAA Timber Engineering Code, AS 1720.

37. Standards Association of Australia (1976). Code of Practice for Installation of Particleboard Flooring, AS 1860.

38. Standards Association of Australia (1978). Methods of Test for Veneer and Plywood, AS 2098.

39. Standards Association of Australia (1979). Structural Plywood, AS 2269.

40. Standards Association of Australia (1979). Plywood and Blockboard for Interior Use, AS 2270.

41. Standards Association of Australia (1979). Glossary of Terms Used in the Plywood Industry, AS 22891

42. Standards Association of Australia (1980). Preservative Treatment of Sawn Timber, Veneer and Plywood, AS 1604.

43. Standards Association of Australia (1980). Flat Pressed Particleboard, AS 1859.

44. Superfesky, M.J., and Lewis, W.C. (1974). Basic properties of three medium density hardboards. USDA Forest Service Research Paper, FPL 238.

45. Superfesky, M.J., Montrey, H.M., and Ramaker, T.J. (1977). Floor and roof sheathing subjected to static loads. Wood Science, Vol.1, No.1, July.

56. Szabo (1982). Flexural properties of waferboard. Technical Report 505 EF, Forintek Canada Eastern Laboratory.

47. US Department of Agriculture, Forest Products Laboratory (1974). Thermal insulation. Report Chapter 20, 'Wood Handbook, Wood as an Engineering Material', USDA Agriculture Handbook No. 72.

48. US Department of Commerce (1974). Product Standard PS 1-74. Washington, DC.

49. US Department of Commerce (1973). Voluntary Product Standard PS 58-73, Basic Hardboard. National Bureau of Standards.

50. World Book - Childcraft International Inc. (1981). The world book encyclopaedia, Vols 23 an 24.

STRUCTURAL PLYWOOD

Lam Pham and Robert H. Leicester[1]

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PART I : STRUCTURAL PLYWOOD

INTRODUCTION

Structural plywood is usually fabricated from rotary peeled veneers. The veneers are normallly glued together so that the grain of adjacent layers is placed at right angles to each other and the peeler checks lie on the inner side of each layer. Plywood panels may consist of plies of different thicknesses, species and grades of wood. The veneers in plywood have slightly different structural properties to the original wood because (i) plywood veneer contains numerous peeler checks which are partly filled with adhesive during the fabrication process; (ii) lower moisture content to which veneers are dried prior to fabrication; and (iii) natural timber defects are dispersed and consequently are less detrimental to strength. Because alternate veneers are laid with the grain at right angles, plywood sheets are much more nearly equal in strength in the longitudinal and perpendicular directions compared with solid timber. Naturally this also means that in any direction, plywood is not as strong as solid timber stressed parallel to the grain. The other advantages of plywood are (i) greater resistance to checking and splitting; (ii) higher shear strength in planes perpendicular to the veneers; and (iii) greater dimensional stability.

In a temperate climate the equilibrium moisture content for a typical solid wood under normal indoor use is about 12% and the moisture content of the wood in outdoor service may be higher than this, possibly around 15%. However, plywood made from the same material would have an initial moisture content of only 10%. As a result, plywood in service tends to be at a lower moisture content than solid wood and its structural properties are correspondingly improved. The effect of moisture content on the properties of plywood have been found to be very similar to that for solid wood.

Part I of this lecture consists of two sections. In the first section, the structural properties of plywood are discussed. In the second section, the design criterion for structural components using plywood are presented.

STRUCTURAL PROPERTIES OF PLYWOOD

Basic Strength and Stiffness

The nature of a strip of plywood is such that the strength and stiffness of the plies that have their grain perpendicular to the direction of stress contribute little to the strength and stiffness of the panel. A good approximation to the structural properties can be achieved by considering 'parallel plies only'. The structural properties of plywood can therefore be computed once the properties of the individual veneers are known. With this approach, plywood can be considered as a collection of pieces of solid timber stressed parallel to the grain. Thus it is appropriate to use the same basic stresses for plywood as are used for solid timber.

When the stress is at 45° to the grain of the face ply and when shear stress is considered, the 'parallel plies only' approach is not appropriate; the calculation of stress is most conveniently based on the full cross-section and the relevant basic stresses are derived directly from experimental data on plywood.

Table 1 summarizes the methods used to calculate structural properties of plywood using the above approach. The following points are noted:

(a) Tension and Compression Strength. For the case of plywood with the grain at 45° to the applied stress, experimental data indicate that the tension strength is about 1/6 and the compression strength is about 1/3 of the strength of the same timber stressed parallel to the grain (compared with an expected value of 1/2.5). The low tension strength is probably due to fracture caused by the differential straining of adjacent veneers.

(b) Bending Strength. It is difficult to calculate the bending strength of plywood because of complexities introduced not only by the layered nature of the material, but also by the nonlinear stress-strain characteristics of the wood in the compression zone. Consequently it is usual in engineering codes to specify that the bending strength M of a plywood strip be computed according to some simple procedure such as the equation presented in Table 1.

The 'parallel plies only' approximation is at its worst when 3-ply plywood is used in bending with the grain of the face veneers perpendicular to the span.

(c) Shear Strength. For plywood there are two shear strength properties that are important in structural design. One is the in-plane or panel shear strength and the other is the interlayer or rolling shear strength.

The panel shear strength is reduced by the presence of peeler checks. This effect is greater for thicker plies and is compensated to some extent by glue infill into the checks. If a veneer contains splits wider than 1.6 mm, it is ineffective in panel shear. An approximate formula for the panel shear strength of plywood without splits is (Curry and Hearmon 1967)

$$F_{sp} = (1.17 - t_v/6.35)F_s + 4.03(n-1)/n.t_v \qquad (1)$$

where F_{sp} is the panel shear strength of the plywood (MPa), F_s is the shear block strength of solid wood (MPa), and n and t_v are the number and thickness of the veneers (mm).

The rolling shear strength of most structural plywoods is approximately the inter-fibre shear strength of the solid wood, but there is a slight increase when the veneers are less than 1.6 mm thick due to glueline effects.

(d) Stiffness. The effect of peeler checks in veneer is to reduce the panel shear stiffness to approximately 80% that of solid wood and the tension stiffness perpendicular to the grain to approximately 70% that of solid wood. The glueline makes a small contribution to stiffness only for plywood that is less than 1.2 mm thick (Curry and Hearmon 1967).

The above presentation of the design information is the traditional North American (and Australian) method. The Europeans use a different approach which gives working stresses related to the full cross-section of the plywood (e.g. CP 112-Part 2:1971). With the full cross-section method, the calculation of geometrical properties is simpler but the working stresses differ for each thickness of plywood and different lay-up of veneers. Consequently a greater number of tables must be used to present the design information.

Both methods ('parallel plies only' and 'full cross-section') are frequently used not only in design codes of practice but also in the presentation of experimental data. The readers are therefore advised to take note of the method used particularly when comparing information from different sources.

Buckling Strength

An important factor that has to be considered in the design of plywood, particularly in engineered structural components, is the buckling of the plywood panels under stress. The design procedure based on small deformation theory involves the use of the elastic buckling load. Good approximation for this load may be obtained from the use of simple variational methods such as those described by Timoshenko (1959). These methods have been applied with the plywood panel considered as an orthotropic plate by Lekhnitskii (1968), March et al. (1942-1949) to solve a variety of problems of plywood buckling. A summary of the plywood plate equations and their applications to buckling problems is given in Part III of this lecture.

Available experimental data indicate that the theoretical elastic buckling load is a good indicator of the start of the growth of large lateral deflection. This is not immediately followed by failure. For most cases, such as in compression and shear, the panel is able to sustain loads much greater than the buckling value. Thus, in design practice the allowable loads must be kept under the buckling loads for the prevention of excessive deformation rather than for strength reasons.

A convenient method of normalizing the information on buckling strength is to define two parameters: a stability factor k_u and a slenderness factor S_o as follows:

$$K_u = F_u/F_{u(s)} \tag{2}$$

and

$$S_o = \sqrt{F_{u(s)}/F_{crit}} \tag{3}$$

where F_u is the ultimate value of the applied stress, $F_{u(s)}$ is the value of F_u for a completely stable system and F_{crit} is the elastic buckling stress of the system.

A typical example for plywood under compression is presented in Figure 4.

The additional strength above the elastic buckling load that may be carried by plywood panel elements is amply demonstrated in the figure. This additional strength can only develop with large deformations and can only be made use of if large deformations are acceptable. No theoretical estimate of post buckling strength is available except for the case of plywood web in shear (Pham and Leicester 1979) and plywood web in compression. These estimates are given in Part IV of this lecture.

The derivation of allowable strength for plywood liable to buckling is identical in approach to the derivation of design buckling strength for solid timber components (which has been described in another lecture).

STRUCTURAL DESIGN

In this section, the design criterion for various structural components using plywood are described.

Plywood as Sheathing Materials

(a) Floors and roofs. Floors and roofs are usually designed for the most adverse effects from a specified concentrated load or a specified uniformly distributed load. The three main design criteria are strength, deformation and vibration. Strength is rarely the critical design criterion for plywood floors and roofs. Current Australian floor design criterion is based on the floor stiffness under a concentrated load. A stiffness of 0.17 mm per 100 N is considered adequate for domestic construction for both deformation and vibration aspects (Mack 1978). A comprehensive report on plywood composite panels for floors and roofs has been compiled by the American Plywood Association (1978).

(b) Shear wall. Plywood sheets, when attached to wall frames, form effective diaphragms to provide shear resistance. The common method of connecting plywood to the wall frame is by nailing. The racking resistance of this form of construction is mostly controlled by the lateral resistance of the nails. For tall panels, the racking deflection may control the design (usually limited to about 8 mm). For large panels with few studs in between, the buckling load of the panel may be the controlling parameter. As mentioned earlier, the design load should be kept below the buckling load to control the growth of lateral deflections on walls which might cause serviceability problems.

Plywood in Engineered Structural Components

(a) Plyweb beams. For both I and box beams, plywood is frequently used as the web material assembled with glue or mechanical fasteners to timber flanges and vertical timber stiffeners at supports and at intervals along the beam span (Figure 5a).

Glued connections, if carried out properly, usually have adequate shear resistance at the interface to ensure complete interaction between the solid timber flange and the plywood web. For the normal design range, buckling of the plywood web under shear is not likely to be a problem. The shear capacity of the plyweb beam in any case is much larger than the buckling shear capacity of the web alone due to the diagonal tension action of the web in the buckling range and the Vierendeel truss action of the beam flanges and stiffeners (Pham 1978). For nailed construction, the load-deformation characteristics of the nail introduce another design criterion for both strength and deflection. This creates no problem for the estimate of strength but deflection is a much more difficult problem. Booth (1974) and Fageiri and Booth (1976) have outlined methods to account for the flange-web joint displacement on the deflection of a plywood beam.

In the computation of deflection of plyweb beams it is essential that the shear deflection be taken into account.

(b) Stressed-skin panels. Plywood stressed-skin panels consist of plywood sheets glued or mechanically fastened to the top or to both top and bottom surfaces of longitudinal timber members. The assembly acts as an integral section to resist bending (Figure 5b). The strength and deflection of a stressed-skin panel are dependent on the strength and rigidity of the joints between the plywood and the longitudinal members. Two special design considerations need to be taken into account: (i) the effect of shear lag on bending stresses and deflection; and (ii) the buckling of the plywood compression skin. The shear lag problem is only significant for relatively short span panels which have the ratio of span to clear spacing less than 8. Foschi (1969a,b) has analysed the problem and has proposed a correction factor to be applied to the bending stresses and deflection calculated from basic engineering formulae applied to the full section.

TABLE 1
STRUCTURAL PROPERTIES OF PLYWOOD

Property	Stress direction with respect to grain direction in face plies	Portion of cross-section to be considered in computing area for moment of inertia	Strength and stiffness (in terms of basic values**)
Tension	Parallel or perpendicular	Parallel plies* only	Basic stress for tension parallel to grain
	±45°	Full cross-sectional area	0.17 x Basic stress for tension parallel to grain
Compression	Parallel or perpendicular	Parallel plies* only	Basic stressin compression parallel to grain
	±45°	Full cross-sectional area	0.34 x Basic stress in compression parallel to grain
Deformation in compression or tension	Parallel or perpendicular	Parallel plies* only	Basic value for modulus of elasticity
	±45°	Full cross-sectional area	0.17 x Basic value for modulus of elasticity
Shear through thickness	Parallel or perpendicular	Full cross-sectional area	Basic shear stress
	±45°	Full cross-sectional area	1.5 x Basic shear stress

* 'Parallel plies' means those plies whose grain direction is parallel to the direction of principal stress.
** For stress computation, basic values can either be ultimate or working values depending on the design method.

Table 1 (continued)

Compression perpendicular to face of plywood	-	Loaded area	Basic stress in compression perpendicular to grain
Strength in bending	Parallel or perpendicular	The bending moment M shall be computed from: $M = g_{19} F'_b (I/y_{max})$ where g_{19} = 1.20 for 3-ply plywood having the grain of the outer plies perpendicular to the span = 0.85 for all other plywood F'_b = basic stress for extreme fibre in bending I = moment of inertia computed on basis of parallel plies only y_{max} = distance from neutral axis to outermost ply having its grain in the direction of the span	Basic stress for extreme fibre in bending
Deflection in bending	Parallel or perpendicular	Deflection may be calculated by the usual formulae, taking as the moment of inertia that of the parallel plies + 0.03 times that of the perpendicular plies	Basic value for modulus of elasticity
Shear deformation in plane of sheet	Parallel or perpendicular	Full cross-sectional area	Basic value for modulus of rigidity
Shear deformation through the thickness	Parallel or perpendicular	Full cross-sectional area	Basic value for modulus of rigidity

TABLE 2
SHEAR IN PLANE OF PLIES

Type of construction	Position of shear	Area to be considered	Stress direction with respect to grain direction in face plies	Permissible stress*	Ultimate stress*
Plywood beams	Longitudinal shear between plies (Fig. 1)	Full shear area	Parallel or perpendicular	0.38 x Basic shear stress	
Box beams and I-beams with plywood webs	Shear between plies of web or between web and flange (Fig. 2)	Area of contact between plywood and flange	Parallel or perpendicular	0.19 x Basic shear stress	
			±45°	0.19 x Basic shear stress	
Panels with plywood covers stressed in compression or tension or both	Shear between plies or between cover and framing members, when (a) depth of member exceeds twice its width and end nogging is used, or (b) depth of member is not more than twice its width and no end nogging is used (Fig. 3)	Area of contact between plywood and framing member (a) Interior members	Parallel or perpendicular	0.38 x Basic shear stress	
			±45°	Basic shear stress	
		(b) Edge members	Parallel or perpendicular	0.19 x Basic shear stress	
			±45°	0.19 x Basic shear stress	

* In terms of basic allowable stress.

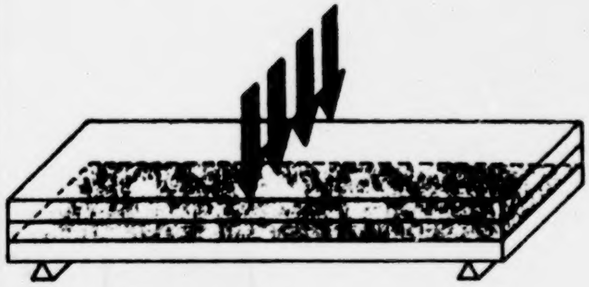


Figure 1 Longitudinal shear between plies

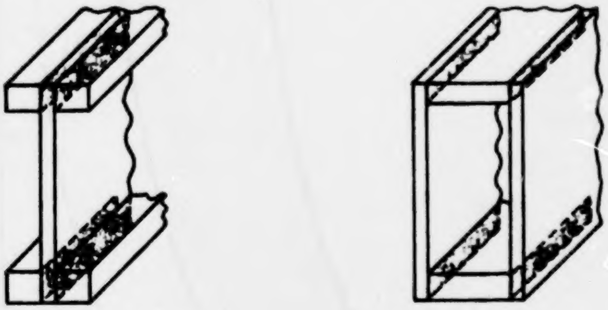


Figure 2 Shear between plies of web and between web and flange

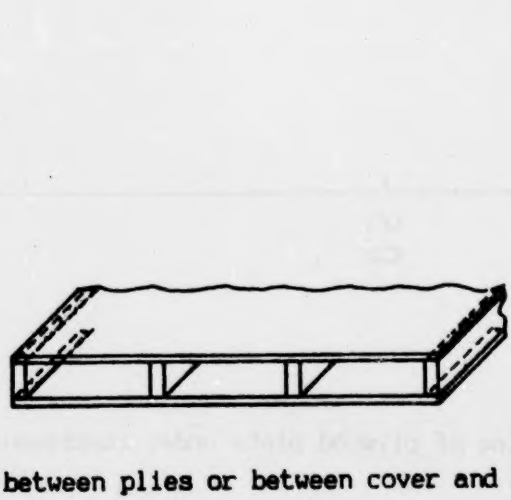


Figure 3 Shear between plies or between cover and framing members

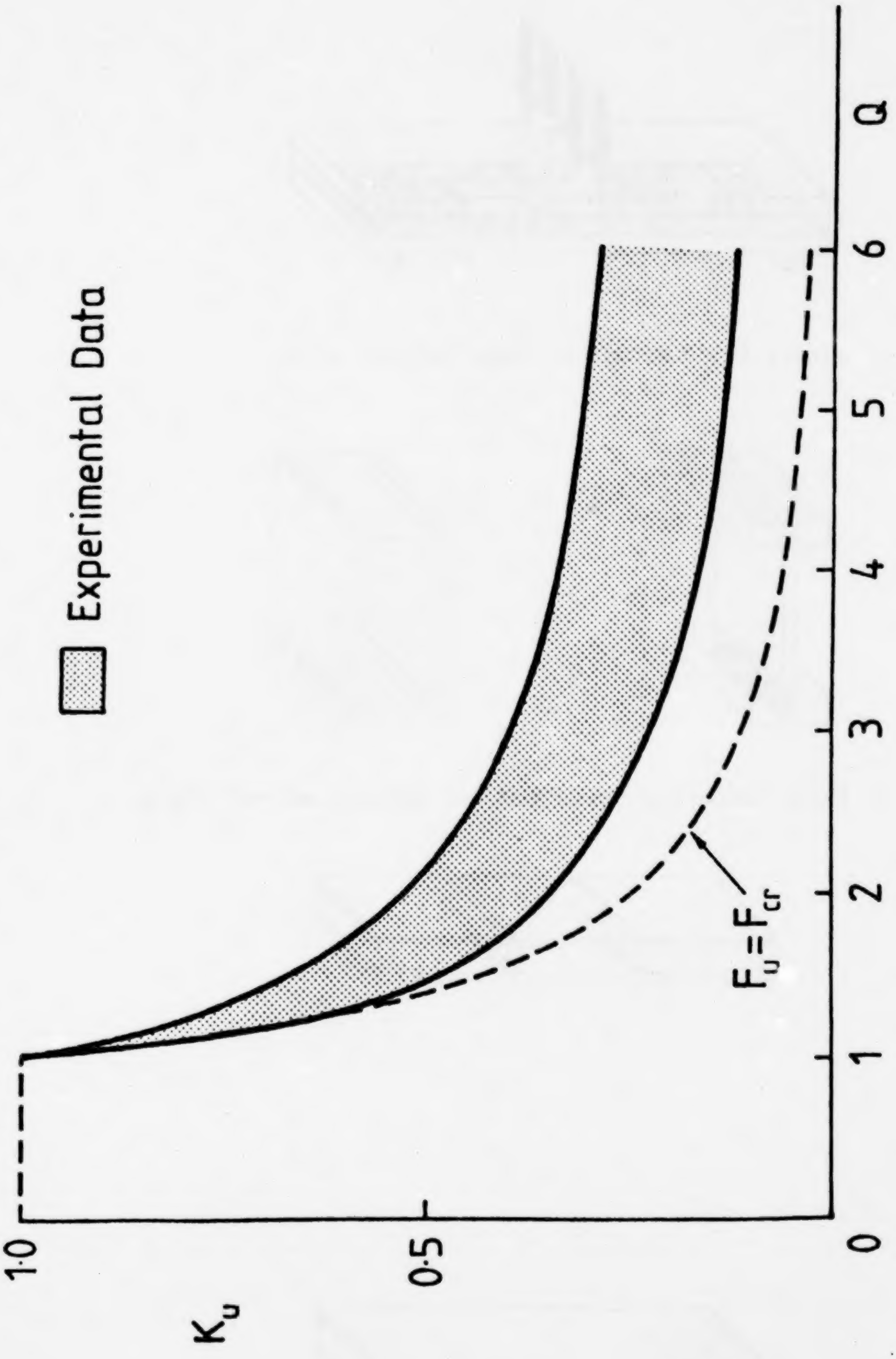
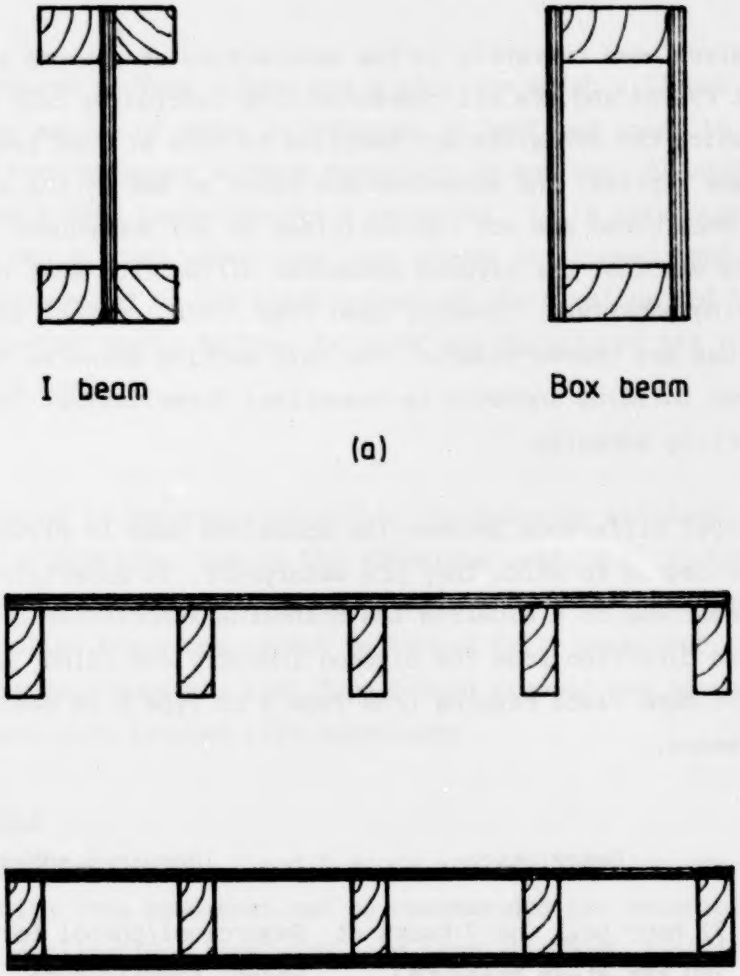


Figure 4 Buckling of plywood plate under compression



I beam

Box beam

(a)

(b) Single and double stress skin panels

Figure 5 Plywood in engineered structural components

PART II : PLYWOOD SPECIFICATION

MATERIAL SPECIFICATION

Plywood Adhesives

The adhesives used currently in the manufacture of plywood are based on synthetic resins and are all thermo-setting (including cold pressed plywood in which the adhesives are modified to cure at room temperature). As the name implies, the adhesives are cured or set by the action of heat and once cured are not replasticised by any subsequent heating. It is in this way that the plywood adhesives differ from most conventional wood working adhesives. Commonly used PVAs, PVCs, contact adhesives and animal glues are thermo-plastic. One cold setting adhesive commonly used in the boat building industry is resorcinol formaldehyde. This is a thermo setting adhesive.

The principal difference between the adhesives used in plywood manufacture is the degree to which they are waterproof. To ascertain the degree of waterproofness of a glueline the Standards Association of Australia, under close direction from the plywood industry and CSIRO, have defined a series of bond tests ranging from Type A to Type D in descending order of performance.

Type of bond	Description	Required adhesive
A	72 hour boil (or 7 hours at 200 kPa steam pressure)	Resorcinol/phenol formaldehyde Phenol formaldehyde Tannin formaldehyde
B	6 hour boil	Urea/melamine formaldehyde
C	3 hour at 70°C	Low extended urea formaldehyde
D	16 to 20 hours cold soak - 20°C	High extension urea formaldehyde

After soaking as prescribed above, the glueline is opened with a testing chisel and visually examined for wood failure. The basic requirement concerning wood failure is that 50% of the failure must occur in the wood for the sample to pass. If this standard is reached the result is a plywood in which the glueline is always equally as strong as the parent wood.

Type A Bond

Plywood manufactured to Type A Bond has a glueline which will not deteriorate under the action of water or extremes of heat and cold. It will withstand long term stresses without degrading in any way. Plywood manufactured to Type A Bond therefore has a permanent, fully waterproof glueline which can be used under long term stress in exposed conditions. It is readily recognised by the black colour of the glueline and the 'Testing PAA Plywood' mark. Marine, Exterior and Structural PAA plywoods have this Type A Bond.

Type B Bond

Type B Bond plywood is incorporated within the exterior standard. However this type of glueline, due to the adhesives used, will in time break down under the action of water or when placed under long term stress. The glueline therefore cannot be termed fully permanent. For example, the standard suggests that Type B Bond plywood can be used for concrete formwork with limited life expectancy.

Type C and D Bond

Plywoods manufactured to Type C and D Bonds are for interior use only. Products made with this bond must not be recommended for exterior use or for structural applications involving long term stresses, even in interior applications. The glueline can be readily recognised by its light colour and the 'PAA Approved Interior Plywood' mark.

Standards

The following standards cover the products produced by the Australian plywood industry and others which are of interest to the consumers of plywood in Australia.

1. Australian Standards

The more important publications of the Standards Association of Australia which refer directly to plywood and which are used in the plywood industry are as follows:

(a) AS 2269 Structural Plywood. This standard specifies requirements for the construction, manufacture, grading and finishing of stress and surface grades of structural plywood. It specifies veneer qualities, bond quality, joints, dimensional tolerances, moisture content and basic working stresses.

The standard prescribes three different methods for the determination of stress grades for structural plywood, viz -

(i) species identification
(ii) density determination; and
(iii) mechanical stress-grading of the finished sheet of plywood.

Two surface grades, based on veneer quality of the face and back veneers, and one bond quality, viz Type A Bond, are prescribed.

Appendices describe sampling, testing and acceptance, stress grading of plywood sheets, physical and mechanical data of structural plywood, stress grades for veneer of individual species, and information to be supplied with enquiries and orders.

(b) AS 2272-1979 Marine Plywood. This standard applies to plywood manufactured for use in the construction of marine craft. Permissible timber species used in the manufacture of this plywood are specified and details are given of quality of veneers, scarf joints and finger joints and manufacturing tolerances. The type of glueline is specified as Type A.

(c) AS 2271-1979 Plywood and Blockboard for Exterior Use. This specification sets out requirements for the construction, manufacture, grading and finishing of plywood and blockboard intended for uses where it is exposed to the weather or damp conditions. It specifies veneer and core strip qualities, bond quality, joints, dimensional tolerances and moisture content. Two bond qualities are specified, viz Type A and Type B Bond.

Exterior grade plywood and blockboard may be grooved, prefinished, overlaid or preservative treated and/or scarf jointed by agreement between the purchaser and the vendor.

Notes: 1. Sampling, testing and acceptance are described in Appendix A.

2. Information required for enquiries and ordering exterior plywood and blockboard are set out in Appendix B.

3. Good practice for handling and storage of plywood and blockboard is described in Appendix C.

(d) AS 2270-1979 Plywood and Blockboard for Interior Use. This standard specifies requirements for the construction, manufacture, grading and finishing of plywood and blockboard intended for non-structural uses where the material is fully protected from the weather or wet or damp conditions. It specifies veneer and core strip qualities, bond quality joints, dimensional tolerances and moisture content. Two bond qualities are specified, viz Type C and Type D Bond.

Interior grade plywood and blockboard may be grooved, prefinished, overlaid or preservative treated and/or scarf-jointed to suit individual needs.

Appendices describe sampling, testing and acceptance, and information to be supplied with enquiries and orders.

Notes: 1. Sampling, testing and acceptance are described in Appendix A.

2. Information required for enquiries and ordering interior plywood and blockboard are set out in Appendix B.

3. Good practice for handling and storage of plywood and blockboard is described in Appendix C.

(e) AS 2098 Methods of Test for Veneer and Plywood. This standard describes in detail the various tests carried out on veneer and plywood. Methods for determination of moisture content, bond quality, resistance of glueline to micro organism attack, quality of scarf joints, depth of peeler checks are given.

(f) AS 2097-1977 Methods for Sampling Veneer and Plywood. This standard describes procedures for the sampling of veneers and plywood and specifies the number of samples required.

Note: Sampling for properties of preservative-treated plywood and veneer is set out in AS 1605 Methods for the Sampling and Analysis of Wood Preservatives and Preservative-treated Wood.

(g) AS 2289-1979 Glossary of Terms Used in the Plywood Industry. This standard is a comprehensive reference manual on all terms used in the plywood industry.

Other Australian Standards used by the plywood industry are as follows:

(a) AS 01-1964 Glossary of Terms Used in the Timber Standards. This standard defines technical and descriptive terms used or likely to be used in the Australian timber standards. This includes terms used in plywood standards.

(b) AS 02-1965 Nomenclature of Australian Timbers. This lists the timbers of Australia arranged in alphabetical order of their standard trade names given their corresponding botanical name, the state in which they occur and other common names. This standard is valuable as there always is a certain amount of confusion over names of species.

(c) AS 1604 Preservative Treated Sawn Timber, Veneer and Plywood. This standard defines the hazard and gives the required penetration patterns and loadings of preservative retentions required for timber and plywood if they are to survive the hazard. This standard also covers the treatment of veneer and plywood to immunize it against lyctus attack.

DESIGN SPECIFICATION

The design rules for plywood are given in AS 1720-1975 Timber Engineering Code. The 'parallel plies only' approach is adopted in this code. A working stress format is used and the basic values of stress are given as basic working stresses. Appendices given in this code allow a quick estimate of the permissible bending moment and stiffness of typical structural plywood as well as the buckling strength of plywood diaphrams. The theoretical bases of the buckling formulae in the code are given in Parts III and IV of this lecture.

PART III : PLYWOOD PLATE EQUATIONS AND EXAMPLES OF THEIR APPLICATIONS

PLYWOOD PLATE EQUATIONS

Introduction

Lateral deflections and buckling strengths are usually the properties of plywood elements that are of most importance in structural design. Consequently in the following, emphasis is placed on the derivation of variational equations that are useful for obtaining approximate estimates of these properties. For simplicity, only plywoods with lay-ups that are symmetrical about the middle surface will be considered.

The notation and sign convention to be used herein is shown in Figure 6. The Cartesian coordinates are taken to lie in the middle surface of the plate along the axes of elastic symmetry, and the coordinate z is taken normal to this surface. The displacement of the middle surface in the x, y and z directions will be denoted by u, v and w. The forces acting on a plywood element of unit width and length are the membrane forces N_x, N_y, N_{xy}, the moments M_x, M_y, M_{xy}, the shears V_x, V_y and the lateral load p.

Plate Forces and Moments

It is shown in standard texts on elasticity that to the degree of approximation required, the strains e_x*, e_y*, $e_{xy}*$ at any point in the plywood plate are related to the displacements u, v, w by

$$e_x* = \frac{\partial u}{\partial x} + \frac{1}{2}\left(\frac{\partial w}{\partial x}\right)^2 - z\frac{\partial^2 w}{\partial x^2}$$

$$e_y* = \frac{\partial v}{\partial y} + \frac{1}{2}\left(\frac{\partial w}{\partial y}\right)^2 - z\frac{\partial^2 w}{\partial y^2} \qquad (4)$$

$$e_{xy}* = \frac{\partial u}{\partial y} + \frac{\partial v}{\partial x} + \left(\frac{\partial w}{\partial x}\right)\left(\frac{\partial w}{\partial y}\right) - 2z\frac{\partial^2 w}{\partial x \partial y}$$

and the corresponding stresses σ_x^*, σ_y^*, σ_{xy}^* are

$$\sigma_x^* = \frac{E_x^*}{\lambda} (e_x^* + \mu_{yx}^* \, e_y^*)$$

$$\sigma_y^* = \frac{E_y^*}{\lambda} (e_y^* + \mu_x y^* \, e_x^*) \qquad (5)$$

$$\sigma_{xy}^* = G_{LT} \, e_{xy}^*$$

with

$$\lambda = 1 - \mu_{TL} \, \mu_{LT} \qquad (6)$$

E_x^*, E_y^* are the Young's moduli in the x and y directions respectively and have the value E_L or E_T. E_L and E_T are the Young's moduli of elasticity parallel and perpendicular to the grain of the plies respectively. μ_{xy}^* and μ_{yx}^* are the Poisson ratios for the plies in the x and y directions respectively and have the value of μ_{TL} or μ_{LT}. μ_{TL} and μ_{LT} are the Poisson ratios for the plies relative to the grain direction and are defined such that

$$E_L \mu_{TL} = E_T \mu_{LT} \qquad (7)$$

The membrane forces and moments are defined by the following equations

$$N_x = \int_{-h/2}^{h/2} \sigma_x^* \, dz$$

$$N_y = \int_{-h/2}^{h/2} \sigma_y^* \, dz \qquad (8)$$

$$N_{xy} = \int_{-h/2}^{h/2} \sigma_{xy}^* \, dz$$

$$M_x = \int_{-h/2}^{h/2} \sigma_x^* \, z \, dz$$

$$M_y = \int_{-h/2}^{h/2} \sigma_y^* \, z \, dz \qquad (9)$$

$$M_{xy} = \int_{-h/2}^{h/2} \sigma_{xy}^* \, z \, dz$$

where h is the thickness of the plywood sheet. Substitution of equations (2) into (5) leads to

$$
\begin{aligned}
N_x &= H_1 e_x + g e_y \\
N_y &= H_2 e_y + g e_x \qquad (10) \\
N_{xy} &= H_{12} e_{xy}
\end{aligned}
$$

where

$$
\begin{aligned}
H_1 &= \int_{-h/2}^{h/2} (E_x{*}/\lambda)\, dz \\
H_2 &= \int_{-h/2}^{h/2} (E_y{*}/\lambda)\, dz \qquad (11) \\
H_{12} &= hG_{LT} \\
g &= hE_L\, \mu_{TL}/\lambda
\end{aligned}
$$

and e_x, e_y, e_{xy} are the middle surface strains obtained from equations (4), i.e.

$$
\begin{aligned}
e_x &= \frac{\partial u}{\partial x} + \frac{1}{2}\left(\frac{\partial w}{\partial x}\right)^2 \\
e_y &= \frac{\partial v}{\partial y} + \frac{1}{2}\left(\frac{\partial w}{\partial y}\right)^2 \qquad (12) \\
e_{xy} &= \frac{\partial u}{\partial y} + \frac{\partial v}{\partial x} + \left(\frac{\partial w}{\partial x}\right)\left(\frac{\partial w}{\partial y}\right)
\end{aligned}
$$

In plate analysis it is convenient to use parameters E_A, E_B, λ_1, λ_2, λ_3 defined by the following equations

$$
\begin{aligned}
E_A &= \left(\frac{\lambda}{h}\right) H_1 \\
&\qquad\qquad (13) \\
E_B &= \left(\frac{\lambda}{h}\right) H_2
\end{aligned}
$$

$$\lambda_1 = \frac{H_1}{H_1H_2 - g^2}$$

$$\lambda_2 = \frac{H_2}{H_1H_2 - g^2} \qquad (14)$$

$$\lambda_3 = \frac{1}{2H_{12}} - \frac{g}{H_1H_2 - g^2}$$

The substitution of equations (5) into (9) leads to

$$M_x = - D_1 \frac{\partial^2 w}{\partial x^2} - \alpha \frac{\partial^2 w}{\partial x \partial y}$$

$$M_y = - D_1 \frac{\partial^2 w}{\partial y^2} - \alpha \frac{\partial^2 w}{\partial x \partial y} \qquad (15)$$

$$M_{xy} = - C \frac{\partial^2 w}{\partial x \partial y}$$

where

$$D_1 = \int_{-h/2}^{h/2} (E_x z^2/\lambda)\, dz$$

$$D_2 = \int_{-h/2}^{h/2} (E_y z^2/\lambda)\, dz \qquad (16)$$

$$C = G_{LT}\, h^3/6$$

$$\alpha = E_L\, \mu_{TL}\, h^3/12\lambda$$

In plate analysis it is convenient to use additional parameters E_1, E_2, D_3 and β defined by the following equations

$$D_1 = E_1 h^3/12\lambda$$

$$D_2 = E_2 h^3/12\lambda$$

$$D_3 = \alpha + C \qquad (17)$$

$$\beta = D_3/\sqrt{D_1 D_2}$$

From equations (16) and (17)

$$D_3 = \frac{2\lambda G_{LT} - E_L \mu_{TL}}{\sqrt{E_1 E_2}} \tag{18}$$

Typical Elastic Parameters

Typical elastic parameters for plywood sheets are given in Tables 3 and 4. The elastic constants used in the computation of these parameters were chosen to conform with the recommendations of AS 1720-1975 and are as follows

$$E_T = E_L/30, \quad G_{LT} = E_L/20, \quad \mu_{TL} = 0.02, \quad \lambda = 0.988$$

Field Equations

For translational equilibrium in the x, y and z direction of the element shown in Figure 6

$$\frac{\partial N_x}{\partial x} + \frac{\partial N_{xy}}{\partial y} = 0 \tag{19}$$

$$\frac{\partial N_{xy}}{\partial x} + \frac{\partial N_y}{\partial y} = 0 \tag{20}$$

$$p + \frac{\partial V_x}{\partial x} + \frac{\partial V_y}{\partial y} + N_x \frac{\partial^2 w}{\partial_x^2} + N_y \frac{\partial^2 w}{\partial_y^2} + 2N_{xy} \frac{\partial^2 w}{\partial x \partial y} = 0 \tag{21}$$

and for rotational equilibrium about the x and y axes

$$V_x = \frac{\partial M_x}{\partial x} + \frac{\partial M_{xy}}{\partial y} \tag{22}$$

$$V_y = \frac{\partial M_{xy}}{\partial x} + \frac{\partial M_y}{\partial y} \tag{23}$$

Equations (15), (17), (21), (22) and (23) lead to the useful field equation

$$D_1 \frac{\partial^4 w}{\partial x^4} + 2D_3 \frac{\partial^4 w}{\partial x^2 \partial y^2} + D_2 \frac{\partial^4 w}{\partial y^4} = p + N_x \frac{\partial^2 w}{\partial x^2} + N_y \frac{\partial^2 w}{\partial y^2} + 2N_{xy} \frac{\partial^2 w}{\partial x \partial y} \tag{24}$$

From equations (9) the following equation of compatibility is obtained

$$\frac{\partial^2 e_x}{\partial y^2} + \frac{\partial^2 e_y}{\partial x^2} - \frac{\partial^2 e_{xy}}{\partial x \partial y} = \left(\frac{\partial^2 w}{\partial x \partial y}\right)^2 - \left(\frac{\partial^2 w}{\partial x^2}\right)\left(\frac{\partial^2 w}{\partial y^2}\right) \tag{25}$$

For the analysis of plate problems it is convenient to introduce an Airy stress function Φ defined as follows

$$\begin{aligned} \frac{\partial^2 \Phi}{\partial x^2} &= N_y \\ \frac{\partial^2 \Phi}{\partial y^2} &= N_x \\ \frac{\partial^2 \Phi}{\partial x \partial y} &= - N_{xy} \end{aligned} \tag{26}$$

It is apparent that membrane stresses derived from this stress function will always automatically satisfy the equations of equilibrium (19) and (20). Substitution of equations (26) into equations (24) and (25) lead to the following field equations that are required for the solution of plate problems according to 'large deformation' theory

$$\begin{aligned} & D_1 \frac{\partial^4 w}{\partial w^4} + 2D_3 \frac{\partial^4 w}{\partial x^2 \partial y^2} + D_2 \frac{\partial^4 w}{\partial y^4} \\ & = p + \left(\frac{\partial^2 \Phi}{\partial y^2}\right)\left(\frac{\partial^2 w}{\partial x^2}\right) + \left(\frac{\partial^2 \Phi}{\partial x^2}\right)\left(\frac{\partial^2 w}{\partial y^2}\right) - 2\left(\frac{\partial^2 \Phi}{\partial x \partial y}\right)\left(\frac{\partial^2 w}{\partial x \partial y}\right) \end{aligned} \tag{27}$$

$$A_1 = \frac{\partial^4 \Phi}{\partial x^4} + 2A_3 \frac{\partial^4 \Phi}{\partial x^2 \partial y^2} + A_2 \frac{\partial^4 \Phi}{\partial y^4} = \left(\frac{\partial^2 w}{\partial x \partial y}\right)^2 - \left(\frac{\partial^2 w}{\partial x^2}\right)\left(\frac{\partial^2 w}{\partial y^2}\right) \tag{28}$$

In small deflection theory, the right hand side of equation (28) is taken to be zero.

Boundary Conditions

The mathematical formulation of boundary conditions that correspond to various practical plate edge conditions are to be found in texts on elastic plate theory. For a straight edge lying along the line $y = 0$, some typical boundary conditions associated with membrane forces are as follows

$$\text{free edge:} \quad N_x = N_{xy} = 0 \tag{29}$$

$$\text{fixed edge:} \quad u = v = 0 \tag{30}$$

and typical boundary conditions associated with the bending moments are as follows

$$\text{free edge:} \quad w = \frac{\partial^2 w}{\partial x^2} = 0 \tag{31}$$

$$\text{simply supported edge:} \quad M_x = w = 0 \tag{32}$$

$$\text{fixed edge:} \quad M_x = V_x - \frac{\partial M_{xy}}{\partial y} = 0 \tag{33}$$

Potential Energy Equation

The strain energy of a deformed plywood plate, denoted by V_p, is defined by

$$V_p = \frac{1}{2} \int_{\text{area}} \int_{-h/2}^{h/2} [\partial_x{}^* e_x{}^* + \partial_y{}^* e_y{}^* + \partial_{xy}{}^* e_{xy}{}^*] \, dz \, dA \tag{34}$$

where dA denotes an elemental area of the middle surface. Substitution of equations (4) and (5) into (34) lead to

$$V_p = V_b + V_m \tag{35}$$

where

$$V_b = \frac{1}{2} \int_{\text{area}} \left\{ D_1 \left(\frac{\partial^2 w}{\partial x^2}\right)^2 + D_2 \left(\frac{\partial^2 w}{\partial y^2}\right)^2 + 2 \left(\frac{\partial^2 w}{\partial x^2}\right)\left(\frac{\partial^2 w}{\partial y^2}\right) + 2C \left(\frac{\partial^2 w}{\partial x \partial y}\right) \right\} dA \tag{36}$$

$$V_m = \frac{1}{2} \int_{\text{area}} \left\{ H_1 e_x{}^2 + H_2 e_y{}^2 + 2g e_x e_y + H_{12} e_{xy}{}^2 \right\} dA \tag{37}$$

The potential energy of the lateral load p, and the boundary membrane forces denoted by Ω_p and Ω_m respectively are

$$\Omega_p = - \int_{area} p.w.\ dA \tag{38}$$

$$\Omega_m = \oint [(N_y v + N_{xy} u)\ dx - (N_x u + N_{xy} v)\ dy] \tag{39}$$

If it is assumed that there is no stretching of the middle surface during bending, i.e. $e_x = e_y = e_{xy} = 0$, then

$$V_m = 0 \tag{40}$$

$$\Omega_m = - \int_{area} [N_x \left(\frac{\partial w}{\partial x}\right)^2 + N_y \left(\frac{\partial w}{\partial y}\right)^2 + 2N_{xy} \left(\frac{\partial w}{\partial x}\right) \left(\frac{\partial w}{\partial y}\right)]\ dA \tag{41}$$

The method for the derivation of equation (41) from equation (39) is described in detail by Timoshenko and Gere.

Solution of the Plate Equations

The plate equations may be solved with the aid of computers by conventional numerical techniques, such as the finite difference or finite element methods. However, for sheets of simple geometrical shape, particularly rectangles, solutions may be more easily obtained through methods associated with mathematical series. These are discussed in detail in texts on elasticity. The following are examples of the use of variational methods to obtain approximate solutions.

LATERAL DEFLECTIONS OF PLYWOOD PLATES

For a plate loaded only by lateral loads, the membrane forces are negligible at small deflections, and from equation (21) the field equation is

$$D_1 = \frac{\partial^4 w}{\partial x^4} + 2D_3 \frac{\partial^4 w}{\partial x^2 \partial y^2} + D_2 \frac{\partial^4 w}{\partial y^4} = p \tag{42}$$

and the appropriate boundary conditions are given by equations (31) to (33).

From equations (35) to (39), the corresponding potential energy functional V is

$$V = \frac{1}{2} \int_{\text{area}} \left\{ D_1 \left(\frac{\partial^2 w}{\partial x^2}\right)^2 + D_2 \left(\frac{\partial^2 w}{\partial y^2}\right)^2 + 2 \left(\frac{\partial^2 w}{\partial x^2}\right) \left(\frac{\partial^2 w}{\partial y^2}\right) + 2C \left(\frac{\partial^2 w}{\partial x \partial y}\right)^2 \right\} dA - \int_{\text{area}} (p.w.)\, dA \qquad (43)$$

An approximate estimate of lateral deflections can be obtained by choosing a reasonable deflection shape for w, substituting into equation (43) and then minimising the functional V.

ELASTIC BUCKLING OF PLYWOOD PLATES

Only the small deformation (critical elastic) buckling of initially flat plates subjected to membrane forces applied at the boundary will be considered. From equation (24) the equilibrium equation for this is

$$D_1 \frac{\partial^4 w}{\partial x^4} + 2D_3 \frac{\partial^4 w}{\partial x^2 \partial y^2} + D_2 \frac{\partial^4 w}{\partial y^4} = N_x \frac{\partial^2 w}{\partial x^2} + N_y \frac{\partial^2 w}{\partial y^2} + 2N_{xy} \frac{\partial^2 w}{\partial x \partial y} \qquad (44)$$

and from equations (35) to (41) the corresponding increment in the potential energy functional V during lateral deformations is

$$\Delta V = \Delta V_b + \Delta \Omega_m \qquad (45)$$

where

$$\Delta V_b = \frac{1}{2} \int_{\text{area}} \left\{ D_1 \left(\frac{\partial^2 w}{\partial x^2}\right)^2 + D_2 \left(\frac{\partial^2 w}{\partial y^2}\right)^2 + 2 \left(\frac{\partial^2 w}{\partial x^2}\right) \left(\frac{\partial^2 w}{\partial y^2}\right) \alpha + 2C \left(\frac{\partial^2 w}{\partial x \partial y}\right)^2 \right\} dA \qquad (46)$$

$$\Delta \Omega_m = - \frac{1}{2} \int_{\text{area}} \left\{ N_x \left(\frac{\partial w}{\partial x}\right)^2 + N_y \left(\frac{\partial w}{\partial y}\right)^2 + 2N_{xy} \left(\frac{\partial w}{\partial w}\right) \left(\frac{\partial w}{\partial y}\right) \right\} dA \qquad (47)$$

where w is the lateral deflection during buckling, and N_x, N_y, N_{xy} are the membrane forces that are present just prior to buckling. The buckling condition is given by

$$\Delta V = 0 \tag{48}$$

EXAMPLES

Deflection of Rectangular Plywood Plates

As a first example, the variational method will be applied to compute the deflection of a rectangular plate laterally loaded by a concentrated load P at the point x_o, y_o. The plate, shown in Figure 7(i), has sides of length a and b. The deflection pattern of the plate will be taken to be approximated by

$$w = A_o \sin\left(\frac{\pi x}{a}\right) \sin\left(\frac{\pi y}{b}\right) \tag{49}$$

where A_o is a constant to be determined. The substitution of equation (49) into (43) leads to

$$V = A_o^2 \frac{ab\pi^4}{8}\left(\frac{D_1}{a^4} + \frac{D_2}{b^4} + \frac{2D_3}{a^2b^2}\right) - P A_o \sin\left(\frac{\pi x_o}{a}\right) \sin\left(\frac{\pi y_o}{b}\right) \tag{50}$$

Hence the minimisation

$$\frac{\partial V}{\partial A_o} = 0 \tag{51}$$

gives

$$A_o = \frac{4P}{\pi^4 ab} \frac{\sin\left(\frac{\pi x}{a}\right) \sin\left(\frac{\pi y}{b}\right)}{\left[\frac{D_1}{a^4} + 2\frac{D_3}{a^2b^2} + \frac{D_2}{b^4}\right]} \tag{52}$$

Single term variational solutions of this type typically under-estimate the true deflection by 5 to 20%. However, they are simple to obtain and are extremely versatile. For example, if the plate is skewed to an angle

θ as indicated in Figure 7(ii), then an appropriate assumption for the deflected shape is

$$w = A_o \sin\left[\frac{\pi}{a}(x - \delta y)\right] \sin\left[\frac{\pi y}{b}\right] \tag{53}$$

where $\delta = \cot\theta$. This leads to the solution

$$A_o = \frac{4P \sin\left[\frac{\pi}{a}(x_o - \delta y_o)\right] \sin\left(\frac{\pi y_o}{b}\right)}{a\, b\, \chi \Pi^4 \sin(\theta)} \tag{54}$$

where

$$\chi = \frac{D_1}{a^4} + 2D_3 \left\{\frac{1}{a^2}\left[\frac{1}{b^2} + \frac{\delta^2}{a^2}\right]\right\} + D_2 \left\{\left[\frac{1}{b^2} + \frac{\delta^2}{a^2}\right]^2 + \frac{4\delta^2}{b^2 a^2}\right\} \tag{55}$$

Uniform End Compression

The variational method will be used to compute the buckling load of a rectangular plywood plate under uniform compression in one direction. A simply supported rectangular plywood plate of length a and width b is subjected to a uniform membrane compression N in the direction of the x-axis as shown in Figure 8. A suitable approximation for the deflection shape is

$$w = A_o \sin\left(\frac{m\pi x}{a}\right) \sin\left(\frac{n\pi y}{b}\right) \tag{56}$$

where A_o is a constant and m and n are integers to be determined. The subsitution of $N_x = N$, $N_y = N_{xy} = 0$ and equation (56) into equations (45) to (47) leads to

$$\Delta V = \frac{ab}{8} A_o^2 \left\{D_1 \left(\frac{m\pi}{a}\right)^4 + D_2 \left(\frac{n\pi}{b}\right) + 2\beta \frac{m^2 n^2 \pi^4}{a^2 b^2} - N \left(\frac{m\pi}{a}\right)^2 \right\} \tag{57}$$

where $\beta = D_3/\sqrt{D_1 D_2}$. Hence the condition $\Delta V = 0$ gives

$$N = \frac{\pi^2 \sqrt{D_1 D_2}}{b^2} \left\{\gamma + 2\beta n_2 + \frac{n^4}{\gamma}\right\} \tag{58}$$

where

$$\gamma = \frac{D_1}{\sqrt{D_2}} \left(\frac{mb}{2}\right)^2 \qquad (59)$$

Obviously the minimum value of N is given for the case m = 1, i.e.

$$N = \frac{\pi^2 \sqrt{D_1 D_2}}{b^2} \left[\gamma + 2\beta + \frac{1}{\gamma}\right] \qquad (60)$$

This equation is plotted in Figure 8. A strip infinitely long in the direction of the x-axis, denoted by N_∞, is obtained from the condition $\delta N/\delta\gamma = 0$ which gives

$$\gamma = 1 \qquad (61)$$

$$N_\infty = \frac{2\pi^2 \sqrt{D_1 D_2}}{b^2} (1 + \beta) \qquad (62)$$

It is apparent from equations (59) and (60) that for an infinitely long strip the plate will buckle into panels of length a_{ch} given by

$$a_{ch} = (D_1 D_2)^{0.25}\, b \qquad (63)$$

This length will be denoted the characteristic buckling length.

For the case of a panel with the sides zero y = 0 and y = b clamped, and the sides x = 0, simply supported, a suitable deflection shape is

$$w = A_o \sin\left(\frac{m\pi x}{a}\right) \left[1 - \cos\left(\frac{2n\pi y}{b}\right)\right] \qquad (64)$$

which leads to the following solution for an infinitely long strip

$$N_\infty = \frac{8\pi^4 \sqrt{D_1 D_2}}{b^2} \left[\frac{1}{\sqrt{3}} + \frac{\beta}{3}\right] \qquad (65)$$

$$a_{ch} = (16\, D_2/3\, D_1)^{0.25}\, b \qquad (66)$$

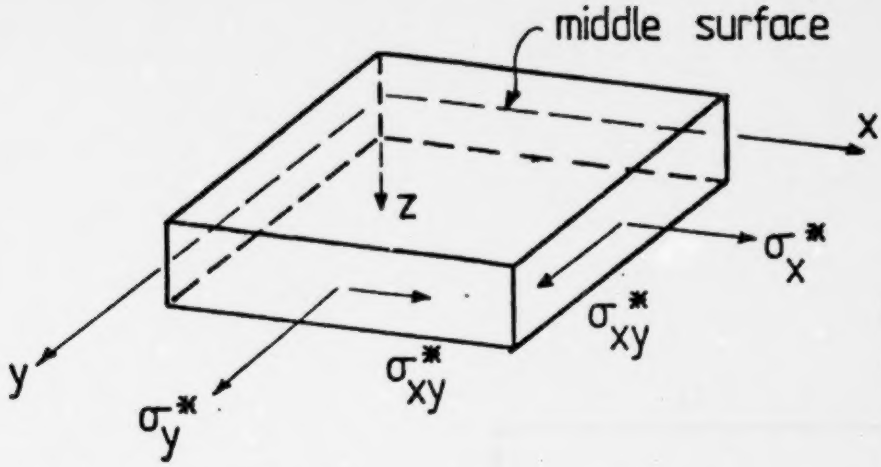
Single term variational solutions of the type derived above typically overestimate the buckling strength by 1 to 10%.

TABLE 3
ELASTIC PARAMETERS FOR PLYWOOD WITH 'EQUAL PLIES' LAY-UP

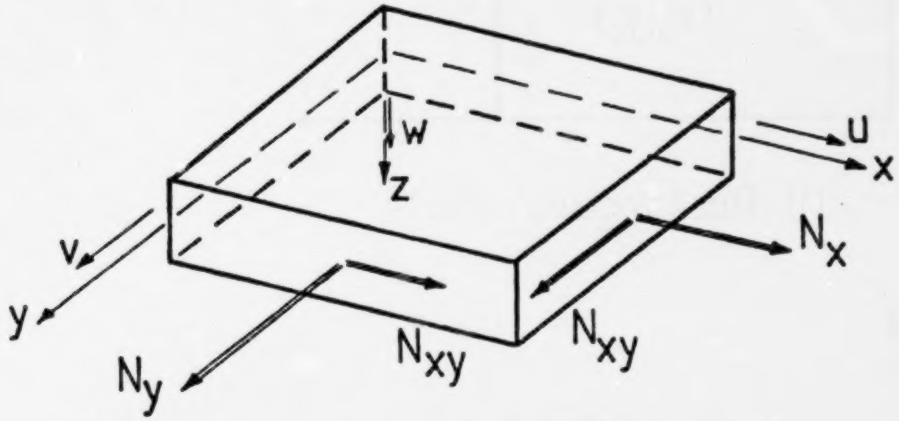
No. of Plies	E_A/E_L	E_B/E_L	E_1/E_L	E_2/E_L	β
3	0.6667	0.3555	0.9642	0.0691	0.4603
5	0.6133	0.4200	0.7989	0.2344	0.2745
7	0.5857	0.4476	0.7210	0.3123	0.2504
9	0.5704	0.4630	0.6765	0.3568	0.2418
11	0.5606	0.4727	0.6478	0.3855	0.2377
Ω	0.5167	0.5167	0.5167	0.5167	0.2300

TABLE 4
ELASTIC PARAMETERS FOR PLYWOOD WITH 'BALANCED PLIES' LAY-UP

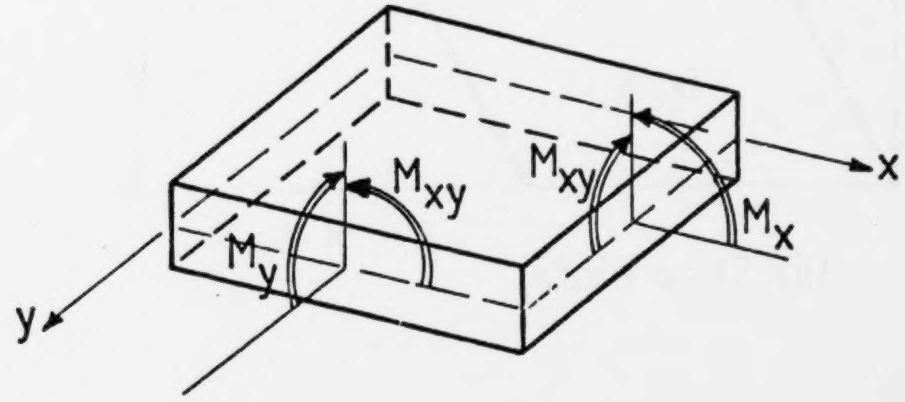
No. of Plies	E_A/E_L	E_B/E_L	E_1/E_L	E_2/E_L	β
3	0.5167	0.5167	0.8792	0.1541	0.3228
5	0.5167	0.5167	0.6073	0.4260	0.2336
7	0.5167	0.5167	0.5569	0.4764	0.2306
9	0.5167	0.5167	0.5393	0.4940	0.2302
11	0.5167	0.5167	0.5312	0.5021	0.2300
Ω	0.5167	0.5167	0.5167	0.5167	0.2300


middle surface
x
z
σ_x^*
σ_{xy}^*
y
σ_{xy}^*
σ_y^*

(a) Coordinate system and stresses

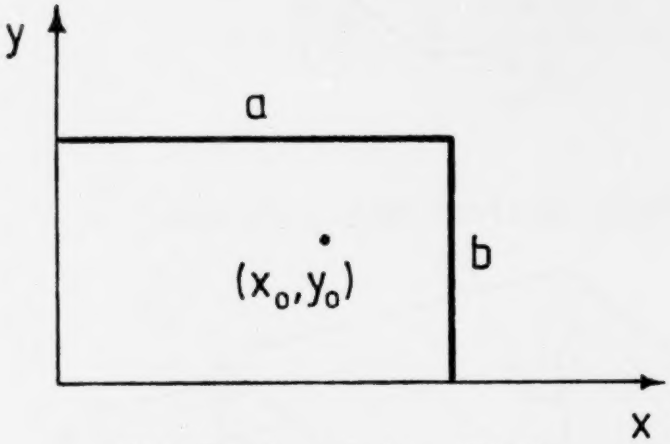

w
z
u
x
v
N_x
y
N_{xy}
N_{xy}
N_y

(b) Displacements and membrane forces

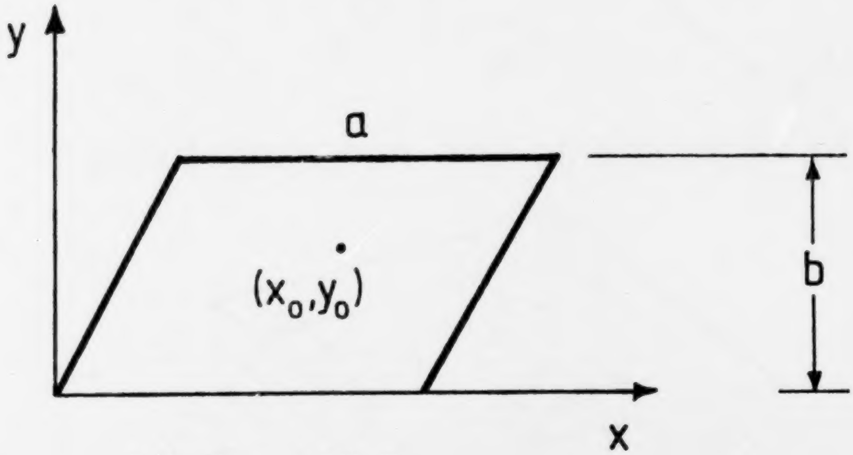

M_{xy}
M_{xy}
x
M_y
M_x
y

(c) Bending and twisting moments

Figure 6 Sign convention for axes, displacement, forces and moments



(i) Rectangular Plate



(ii) Skew Plate

Figure 7 Notation for plates

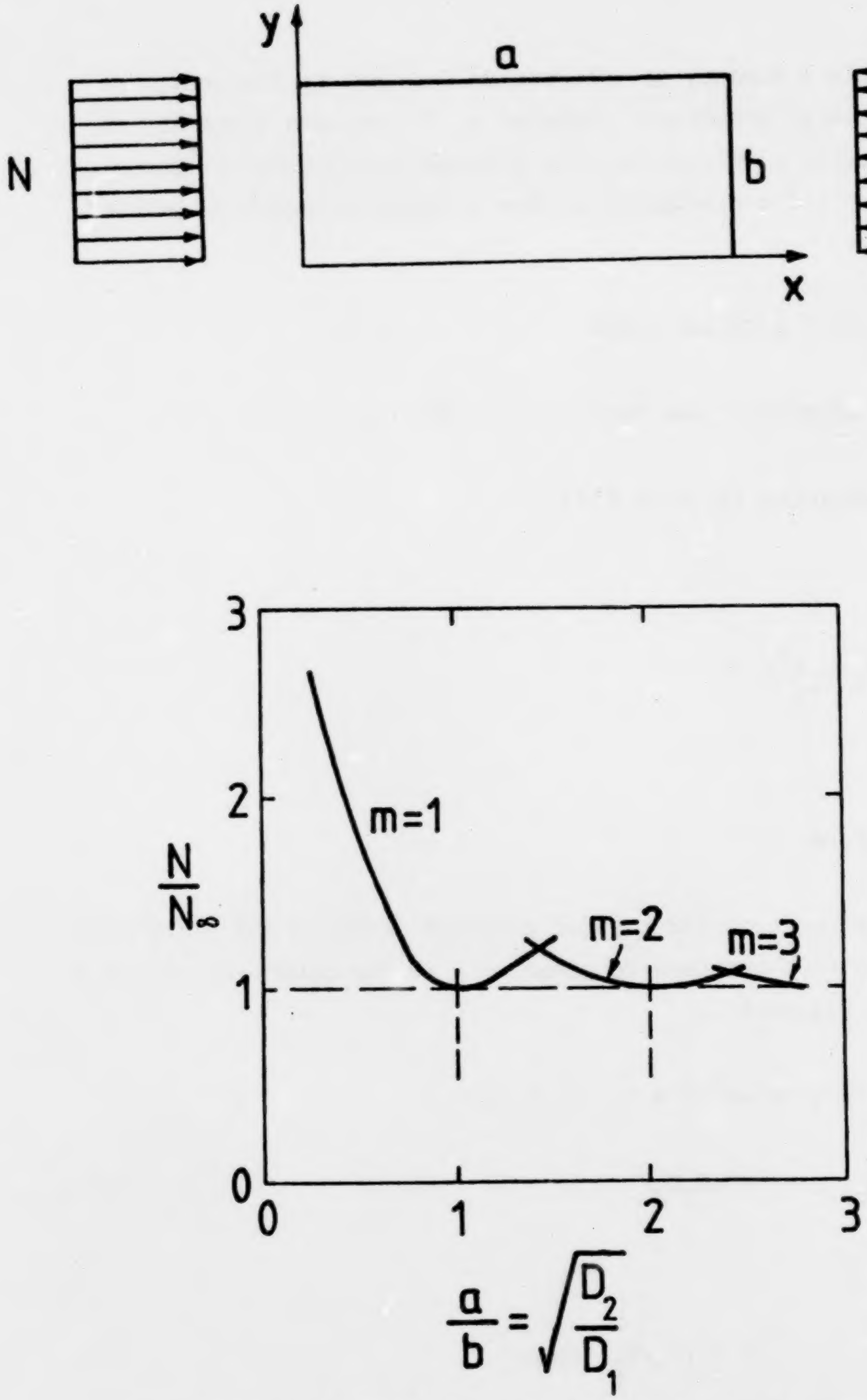


Figure 8 Buckling of plywood plate under uniform compression

PART IV : STRENGTH OF PLYWOOD PLATES

INTRODUCTION

This part presents a summary of information relevant to the design of plywood in engineered structural components. It includes a summary of formulae for elastic buckling loads of plywood plates under compression, bending and shear and a discussion on the ultimate strength of buckled plywood plates.

FORMULAE FOR ELASTIC BUCKLING LOADS

Definitions and Notations (see Part III for details)

$D_1 = E_1h^3/12\lambda$ (Equation 17, Part III)

$D_2 = E_2h^3/12\lambda$

$D_3 = G_{LT}h^3/6 + E_L\mu_{TL}h^3/12\lambda$

$\beta = D_3/\sqrt{D_1D_2}$

Uniform and Compression

A simply supported rectangular plywood plate of length a and width b is subjected to a uniform membrane compression N in the direction of the x-axis as shown in Figure 8.

The elastic buckling value of N is given by

$$N_{crit} = (\pi^2\sqrt{D_1D_2}/b^2)\ [\gamma + 2\beta + (1/\gamma) \tag{67}$$

where

$$\gamma = \sqrt{D_1/D_2}\ (mb/a)^2 \tag{68}$$

with m = 1, 2, 3.....

For a strip infinitely long in the direction of the x-axis, the elastic buckling load, N_∞, is given by

$$N_\infty = (2\pi^2 \sqrt{D_1 D_2}/b^2)\ (1 + \beta) \tag{69}$$

It is apparent from equations (67) and (68) that for an infinitely long strip the plate will buckle into panels of length a_{ch} given by

$$a_{ch} = (D_1\ D_2)^{0.25}\ b \tag{70}$$

This length is denoted the characteristic buckling length. For the case of a panel with sides y = 0 and y = b clamped, the buckling load for an infinitely long strip is

$$N_\infty = (8\pi^4 \sqrt{D_1 D_2}/b^2)(0.577 + \beta/3) \tag{71}$$

and the characteristic buckling length

$$a_{ch} = (16\ D_2/3\ D_1)^{0.25}\ b \tag{72}$$

Edgewise Bending

A simply supported rectangular plate of length a and width b as shown in Figure 9 is loaded in edgewise bending so that the membrane stresses prior to buckling are given by

$$N_x = N[(2y/b) - 1]$$

$$N_y = N_{xy} = 0 \tag{73}$$

The value of the buckling force parameter N is

$$N_{crit} = (9\pi^4/32)(\sqrt{D_1 D_2}/b^2)\ \sqrt{[\gamma+\beta+(1/\gamma)][\gamma+8\beta+(1/\gamma)]} \tag{74}$$

where

$$\gamma = (mb/a)^2\ \sqrt{D_1/D_2} \tag{75}$$

The solution for an infinitely long plate is

$$N_{\infty crit} = 3\pi^4\ \sqrt{D_1 D_2}\ \sqrt{0.25 + 40\beta + 16\beta^2} \tag{76}$$

and the characteristic buckling length

$$a_{ch} = (D_1/4D_2)^{0.25}.b \qquad (77)$$

Edge Shear

An infinitely long simply supported strip of width b is subjected to edge shear force N_∞ as shown in Figure 10. For these conditions the plate buckles roughly in skew-shaped panels with the buckling force

$$N_{\infty crit} \cong (\pi^2\sqrt{D_1D_2}/b^2)(D_2/D_1)^{0.25}(3.66 + 2.0\beta) \qquad (78)$$

and the characteristic buckling length

$$a_{ch} \cong (D_1/D_2)^{0.25}\ (1.0 + 0.22\beta).b \qquad (79)$$

Combined Edge Loads

For an infinitely long simply supported strip of plywood subjected to a uniform end compression N_x, an edgewise bending with maximum value N_{bx} and an edge shear N_{xy}, an appropriate criterion for the onset of buckling obtained from the result of numerical analysis is

$$(N_x/N_{xo}) + (N_{xy}/N_{xyo})^2 + (N_{bx}/N_{bxo})^2 = 1 \qquad (80)$$

where N_{xo}, N_{xyo} and N_{bxo} are the elastic buckling values of N_x, N_{xy} and N_{bx} respectively if these forces were acting alone.

ULTIMATE STRENGTH OF BUCKLED PLYWOOD PLATES

If large deformations are acceptable in a structural design, it is often very advantageous to make use of the additional strength above the elastic buckling load that may be carried by plywood plates. The necessary large deformation theory for this is outside the scope of this lecture, however two particular structural cases will be discussed to provide some insight on large deformation behaviour.

Strength of Plywood Plate in End Compression

For the thin plate loaded in end compression as shown in Figure 11, large deformation theory shows that with increasing deformations the load tends to be carried by two edge strips of width denoted $b_o/2$. If it is assumed that just prior to failure the membrane forces on the edge strip are $N_{u(s)}$, the maximum that would be possible for completely stable members, then the average membrane edge force N_u is given by

$$N_u = N_{u(s)} \, (b_o/b) \qquad (81)$$

For the particular plywood sheet, the elastic buckling load is

$$N_{cr} = (\pi^2 \sqrt{D_1 D_2}/b^2) \, k_o \qquad (82)$$

Furthermore, if it is assumed that the plate acts effectively as a strip of width b_o in elastic buckling, then

$$N_u = (\pi^2 \sqrt{D_1 D_2}/b_o^2) \, k_o \qquad (83)$$

Hence

$$N_u/N_{u(s)} = b_o/b = \sqrt{N_{cr}/N_{u(s)}} \qquad (84)$$

or

$$N_u = \sqrt{N_{cr} N_{u(s)}} \qquad (85)$$

Equation (85) must be considered to be merely an estimate of the panel strength, not only because of the heuristic arguments used to arrive at it, but also because the magnitudes of initial imperfections and other important strength parameters have not been considered. Figure 12 is a graph of the results of a series of tests on plywood sheets by March _et al._ (1945). It indicates that equation (85) is valid for extremely thin plates ($N_{cr}/N_{u(s)}$<0.1), but that a more suitable formula for design is

$$N_u = 0.7 \sqrt{N_{cr} N_{u(s)}} \qquad (86)$$

Strength of Plywood Web in Shear

Consider a slender plywood panel loaded in shear (Figure 13), prior to buckling the web panel develops both tensile and direct compressive stresses. After buckling has developed, the web has no further capacity to carry extra compressive stresses but further load can still be carried by diagonal tension. Many models for this diagonal tension have been developed to predict the post-buckling load capacity but are not suitable for application to plywood web beams. For example, the partial diagonal tension model (Porter et al. 1975) currently being used for steel plate girders has been found to be unsuitable because plywood webs do not display the required ductile yield plateau, and because timber flanges are, relatively speaking, too thick to form plastic hinges. Of the various models available, the complete diagonal tension model (Kuhn 1956) appear to best fit the experimental behaviour observed in tests on plywood-web beams, and this model will be used in this lecture for the ultimate shear strength of plywood webs.

The ultimate strength V_u of a plywood-web beam will be taken to be the lesser of the following:

$$\begin{aligned} V_{u1} &= V_e + V_d + V_f \\ V_{u2} &= V_s + V_f \end{aligned} \qquad (87)$$

where V_e is the elastic shear buckling load of a plywood web
V_d is the shear carrying capacity resulting from diagonal tension strength of the plywood web
V_f is the contribution of the flanges and web stiffeners to the shear carrying capacity of the beam
V_s is the panel shear strength of a stable plywood web.

The method used to obtain the elastic buckling load V_e has been described in Part III. The stable shear strength V_s, which is the ultimate panel shear strength in the absence of buckling, has been described in Part I of this lecture.

The interaction between the web and the flanges is complex, particularly in the post-buckling region and will not be discussed in detail herein. Roughly, the contribution of the flanges to the shear carrying capacity for the test beams, denoted by V_f, is estimated by assuming that the flanges act as simply-supported beams with deflection matching those of the plywood web under shear.

Finally the shear force V_d carried by a diagonal tension field N_d inclined at an angle θ to the flanges is given by

$$V_d = 1/2a.N_d\sin\theta.\cos\theta \qquad (88)$$

where N_d is the tension membrane force per unit length (Figure 13).

The tension membrane force N_d has a maximum possible value given by

$$N_d = N_{t\theta} - N_{tE} \qquad (89)$$

where $N_{t\theta}$ is the membrane force of the plywood in the direction θ, and N_{tE} is the membrane buckling force N_E resolved in the direction θ, i.e.

$$N_{tE} = 2N_E\sin\theta\cos\theta \qquad (90)$$

The membrane force $N_{t\theta}$ can be obtained by fitting a curve to the following three points of known tensile strength (Figure 14)

$$\begin{array}{ll} \theta = 0 & N_{t\theta} = k.t_w.f_{tu} \\ \theta = \Pi/4 & N_{t\theta} = t_w.F_{tu}/6 \\ \theta = \Pi/2 & N_{t\theta} = (1-k)t_w.F_{tu} \end{array} \qquad (91)$$

where F_{tu} is the ultimate tensile stress of the timber along the grain, k is the ratio of areas of effective plies to the gross area and t_w is the plywood web thickness.

The expression for V_d is taken so as to give the highest possible shear capacity. This is achieved when θ is approximately equal to 15^o. For this case, equations (88), (89) and (90) lead to

$$V_d = 1/4a\ (N_{t\theta} - N_e/2) \qquad (92)$$

Figure 15 is a graph of the results of a series of tests on plywood web beams in shear. It indicates that the above method of estimating plywood web ultimate shear strength agrees fairly well with available experimental data.

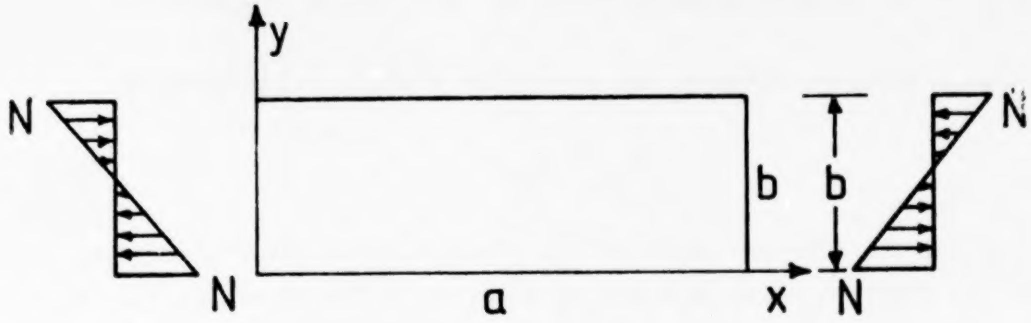


Figure 9 Plate under edgewise bending

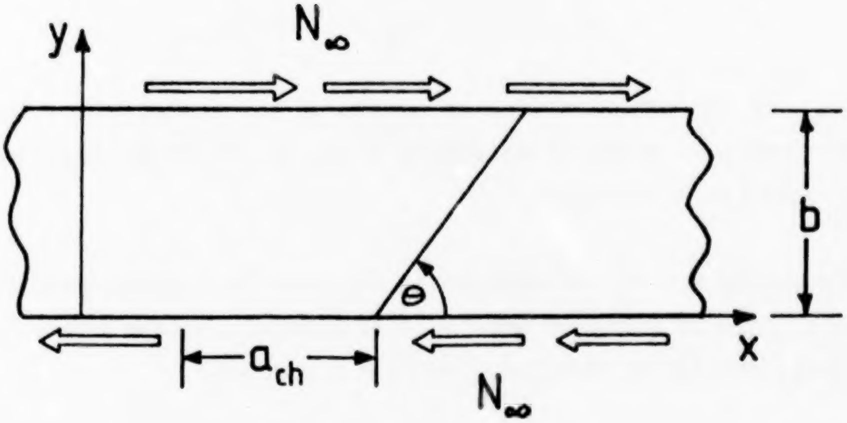
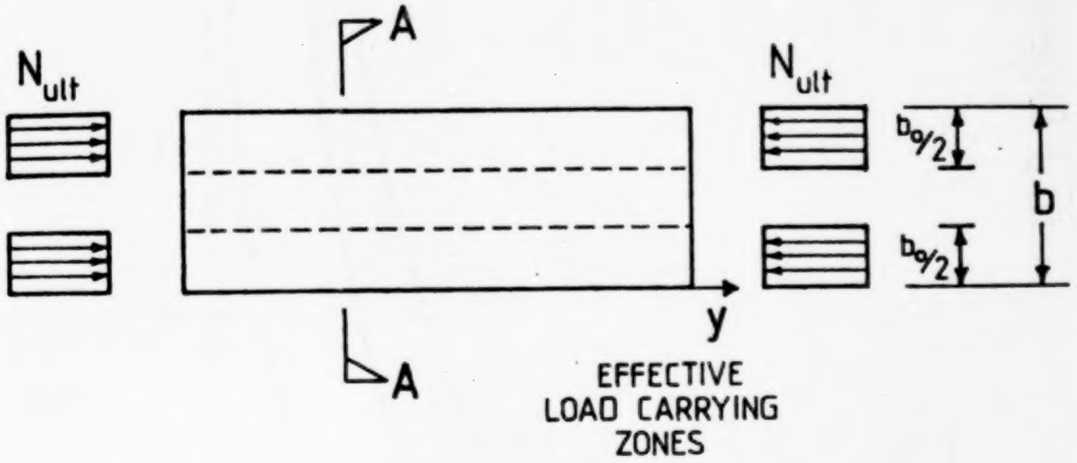
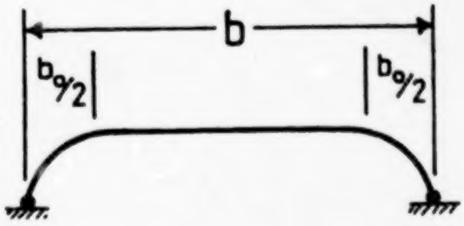

y
N_∞
b
θ
a_{ch}
x
N_∞

Figure 10 Plate under shear


A
N ult
N ult
b o/2
b
b o/2
y
A
EFFECTIVE
LOAD CARRYING
ZONES

(i) Plan



(ii) Section

Figure 11 Ultimate strength of plate under compression

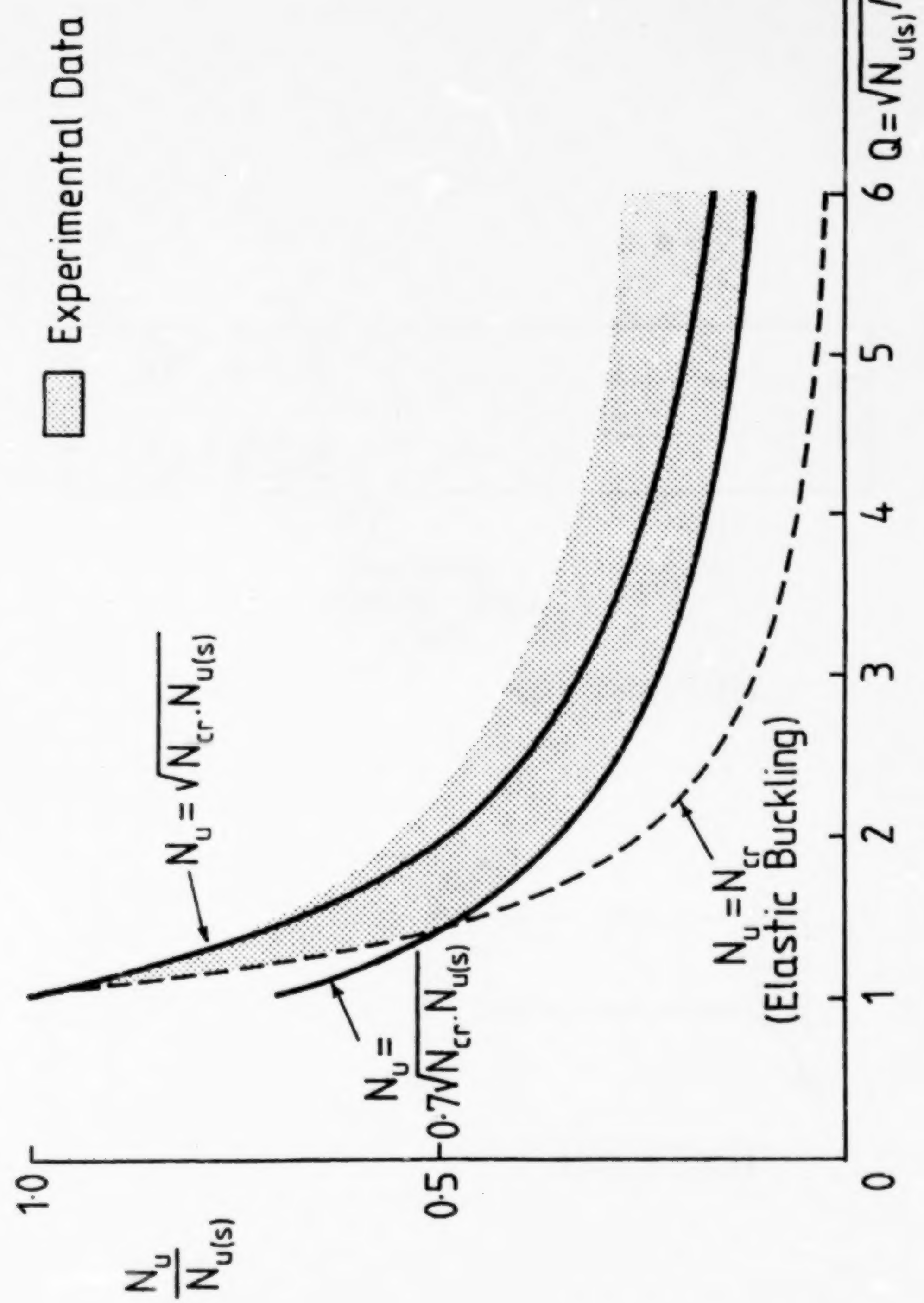


Figure 12 Experimental data for ultimate strength of plate under compression

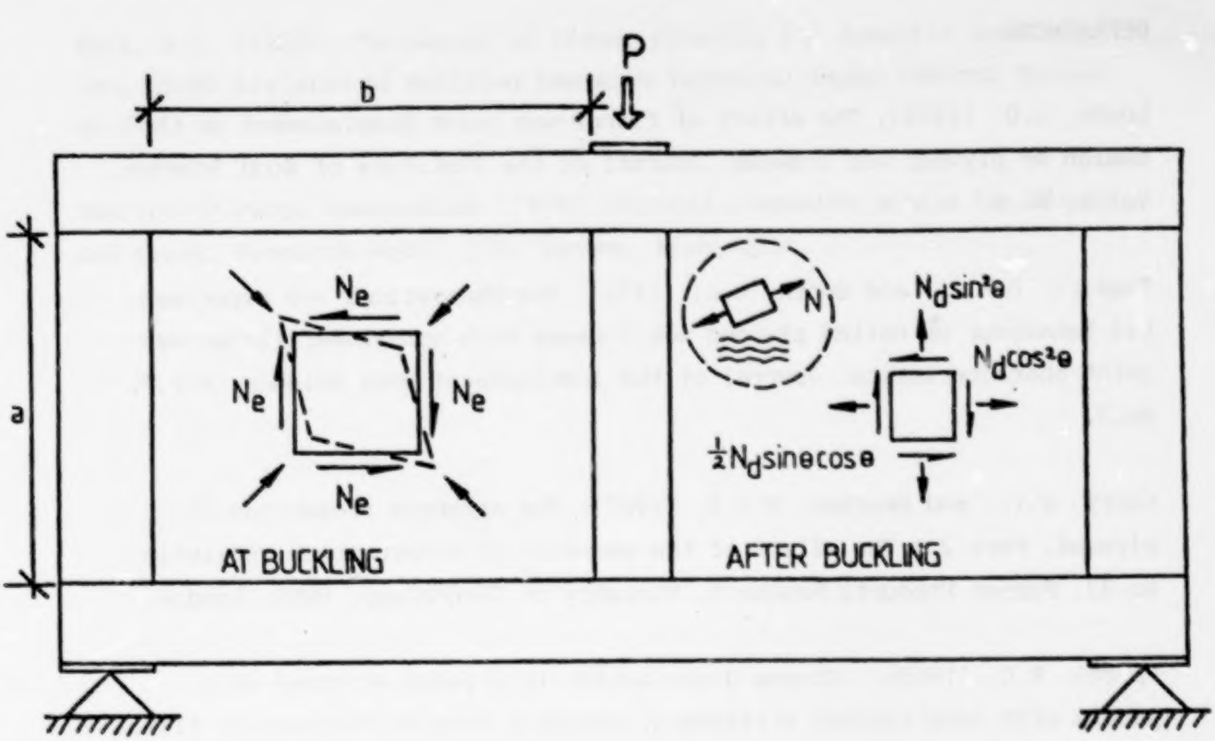

P
b
a
N_e
N_e
N_e
N_e
AT BUCKLING
N
N
$N_d \sin^2\theta$
$N_d \cos^2\theta$
$\frac{1}{2}N_d \sin\theta\cos\theta$
AFTER BUCKLING

Figure 13 Ultimate strength of plate under shear

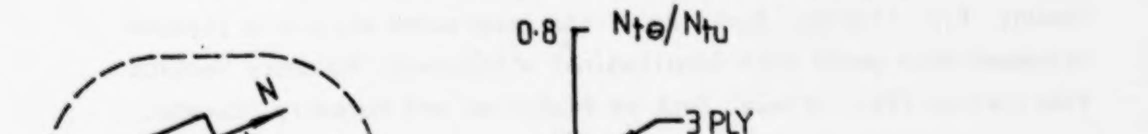


Figure 14 Variation of strength with direction of tensil stress for plywood plate

REFERENCES

Booth, L.G. (1974). The effect of flange-web joint displacement on the design of plywood web I-beams. Journal of the Institute of Wood Science, Vol.6, No.6.

Fageiri, O.M.E., and Booth, L.G. (1976). The theoretical and experimental behaviour of nailed plywood web I-beams with non-linear flange-web joint characteristics. Journal of the Institute of Wood Science, Vol.7, No.3.

Curry, W.T., and Hearmon, R.F.S. (1967). The strength properties of plywood. Part 2 - The effect of the geometry of construction. Bulletin No.33. Forest Products Research, Ministry of Technology, HMSO, London.

Foschi, R.O. (1969a). Stress distribution in plywood stressed-skin panels with longitudinal stiffeners. Forestry Service Publication 1261. Ottawa, Dept of Fisheries and Forestry, Canada.

Foschi, R.O. (1969b). Buckling of the compressed skin of a plywood stressed-skin panel with longitudinal stiffeners. Forestry Service Publication 1265. Ottawa, Dept of Fisheries and Forestry, Canada.

March, H.W. et al. Buckling of flat plywood plates in compression, shear, or combined compression and shear. Reports No. 1316 (April 1942), 1316-A (September 1942), 1316-B (November 1942), 1316-C (January 1943), 1316-D (June 1943), 1316-E (October 1943), 1316-F (October 1943), 1316-G (November 1943), 1316-H (July 1950, Revised), 1316-I (March 1945), 1316-J (February 1949). Forest Products Laboratory, USDA Forest Service, Madison, Wisconsin, USA.

Timoshenko, S.P., and Gere, J.M. (1961). Theory of elastic stability. McGraw-Hill Book Co. Inc., New York.

Lekhnitskii, S.G. (1968). Anisotropic plates. (English Translation by Tsai, S.W., and Cheron, T.). Gordon and Breach, New York.

Mack, J.J. (1968). The design of timber flooring for domestic construction. CSIRO Division of Building Research Technical Paper (Second Series) No.24.

American Plywood Association (1978). Plywood composite panels for floors and roofs. Research Report 135, Tacoma, Washington.

GLUED LAMINATED TIMBER

Robert H. Leicester[1/]

1. INTRODUCTION

Glued laminated timber, or 'glulam' as it is often referred to, is formed by gluing together thin laminae to form structural members, often of large size. In Australia, a great variety of species, including both softwoods and hardwoods, have been used for glulam. Each species has its own particular value in this regard. Softwood glulam tends to be easier to fabricate and therefore will often by the most economical type to use, while hardwood glulam is often favoured because of the specific properties of its timber such as its aesthetics or natural durability.

The glulam laminae used commercially ranges in thickness from 50 mm down to veneer thickness of 5 mm or even less. In order to form laminae of sufficient length, particularly for large members, it is usually necessary to 'end-joint' planks of timber. Because glulam members are fabricated from small elements, they can be of any size or shape. Members longer than 30 m and deeper than 2 m have been fabricated. Most commonly, members are rectangular in cross-section and are either straight or have a uniform curvature.

2. STRENGTH THEORIES FOR GLULAM

Early theories of glulam strength were related to the 'I_K/I_G' concept illustrated in Figure 1. The term I_K refers to the moment of inertia of the projected area of all knots within 150 mm of the cross-section under scrutiny; the term I_G denotes the moment of inertia of the gross cross-section. The I_K/I_G concept is based on finding the correlation of strength with the parameter $1 - I_K/I_G$. The 5 percent exclusion limit is then used to predict the characteristic strength R_K. By running surveys of knot sizes in timber boards, it is possible to predict the statistics of I_K/I_G values that would be expected for various glulam lay-ups.

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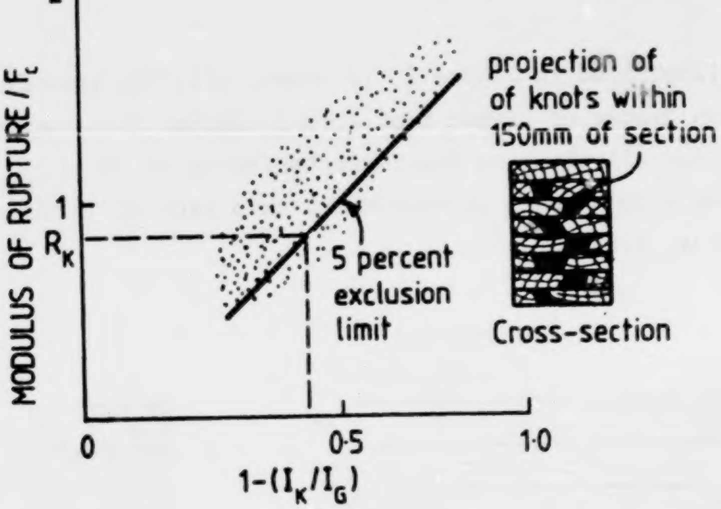


Figure 1   Illustration of the knot ratio concept

The I_K/I_G concept has been used extensively (e.g. Nemeth 1967) but is not particularly effective for covering a wide range of parameters (Bohannon and Moody 1973). Accordingly other strength theories have been proposed. The following will contain a description of the theory used for the Australian Standard AS 1720 (Standards Association of Australia 1975). This theory is based on the concept that the strength of glulam is equal to the strength of solid wood, with some slight enhancement due to one of the following two reasons:

- there is a 'local reinforcement' effect wherein the weakness of a local defect is overcome to some extent by the assistance of clear wood in the laminae on each side of the defect

- there is a 'load sharing' effect wherein the 5 percentile or characteristic strength of glulam is increased because the joint probability of occurrence of several weak laminae is less than that of a single piece of solid wood of comparable weakness.

3. THEORY OF LOCAL REINFORCEMENT

3.1 Strength of Butt-Jointed Glulam

A butt joint, illustrated in Figure 2, is essentially the absence of an end-joint between pieces of timber that form a lamina. This type of joint is useful for illustrating the characteristics of local reinforcement, as without such reinforcement this type of joint would have no strength at all.

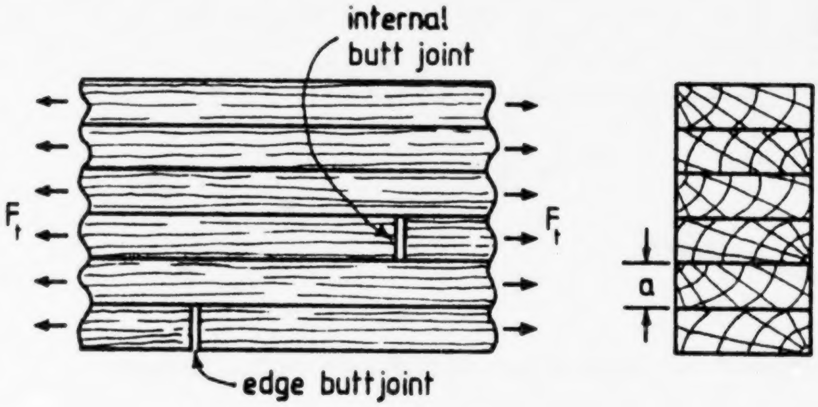


Figure 2 Butt joints in glulam

The strength of butt joints has been discussed earlier. The applied load is stated in terms of a stress intensity factor K_I which is given by

$$K_I = f_t \; (\pi a)^{0.5} \qquad (1)$$

for internal butt joints. The applied nominal tension stress f_t is stated in MPa units and the lamina thickness a is stated in millimetres. For edge butt joints the intensity factor is about 40 per cent greater than the value given by equation (1).

The critical stress intensity factor can be conservatively estimated from

$$K_{IC} = 0.15 \; \rho \qquad (2)$$

where ρ is the density of air dry timber in kg/m^3 units.

Equations (1) and (2) lead to the value of stress to cause fracture to be given by

$$f_{t(ult)} = 0.15\ \rho/(\pi a)^{0.5} \qquad (3)$$

If there are N butt joints located in the zone of maximum stress, then a 'weakest-link' situation applies, and the stress to cause fracture of the member is reduced by a factor N^{α}, where α is the coefficient of variation of the butt joint strength. Typically $\alpha = 0.2$ and hence equation (3) should be modified to read

$$f_{t(ult)} = 0.15\ \rho/(\pi^{0.5}\ a^{0.5}\ N^{0.2}) \qquad (4)$$

It should be noted that equation (4) indicates that the nominal tension stress to cause failure increases as the lamina thickness a decreases. For a lamina thickness of a few millimetres the tension strength is close to that of structural grade timber; however for lamina thicknesses in the range that is most frequently used in most fabrication, say 20 to 40 mm, the fracture strength may be less than a quarter the value of the tension strength of structural timber.

As would be expected from the above, glulam fabricated from veneer is extremely strong (Bohlen 1972, 1974; Koch 173; Koch and Woodson 1968). Such glulam is often referred to as 'microlam' or 'laminated lumber veneer'.

3.2 Strength of Continuous Laminae

Figure 3 illustrates schematically the method whereby the stress in a lamina, interrupted by the discontinuity of a defect, may be carried by the adjacent laminae provided these are straight grained material that is defect-free. This is the same action as that associated with a butt joint; however in the case of a natural defect the reinforcement effect is more efficient as the defect is not as sharply discontinuous as a butt joint.

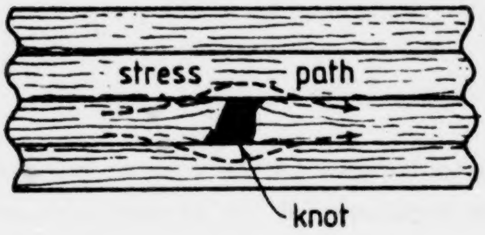


Figure 3 Local reinforcement of a defect

The method for assessing the strength of glulam is illustrated in Figure 4. This evaluation is made in terms of the grade ratio of the timber used for the laminae; the grade ratio, denoted by GR, is defined as follows

$$GR = R_{kg}/R_{kc} \quad (5)$$

where R_{kg} is the 5 percentile characteristic value of bending strength of the graded timber and R_{kc} is the characteristic bending strength of small clear wood specimens.

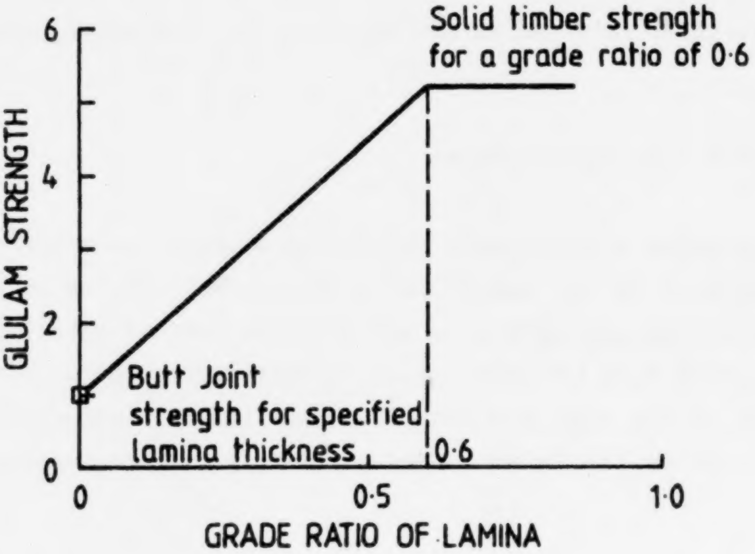


Figure 4 Method of establishing glulam strength

The evaluation of glulam strength is made in the following steps:

(a) The strength of butt jointed glulam is computed and taken to be the value relevant to a grade ratio of zero.

(b) The strength of clear structural timber is determined and taken to be relevant to a grade ratio of 0.6.

(c) The glulam strength for the grade ratio relevant to the laminae used is obtained by interpolation between the values for grade ratios of zero and 0.6.

The above applies to bending and tension strength. It is assumed that the reinforcement effect on compression strength is negligible.

Examples of the reinforcement factor specified in the Australian Standard AS 1720 are given in Table 1.

TABLE 1
FACTOR FOR LOCAL REINFORCEMENT
(After Standards Association of Australia 1975)

Lamina thickness (mm)	Reinforcement factor*			
	Select engineering grade	Standard engineering grade	Select engineering grade	Standard engineering grade
50	1.00	1.10	1.15	1.20
20	1.00	1.15	1.25	1.40
5	1.00	1.25	1.45	1.70

* Reinforcement factor is only applicable to straight grained softwoods

4. THE LOAD SHARING EFFECT

The load sharing effect of laminae constrained to deform together will be discussed elsewhere. It is a function of the interaction of the load-deformation characteristics of randomly selected groups of laminae. The enhancement of the 5 percentile strength depends on the species considered and the following is a typical set of values:

Number of laminae	Load sharing factor
1	1.00
2	1.14
5	1.26
10	1.33

In the Australian Standard AS 1720 (Standards Association of Australia 1975) the effective number of laminae for load sharing estimates is specified as the following:

- tension member ... N
- column ... 0.5 N
- beam ... 0.25 N

where N is the total number of laminae in the member.

5. STIFFNESS OF GLULAM

It may be assumed that laminating does not affect the stiffness properties of the laminae. If the laminae are of mixed species or grades, then a conventional analysis using the transformed section technique may be used to evaluate the deflections of a glulam beam or the extension of a glulam tie.

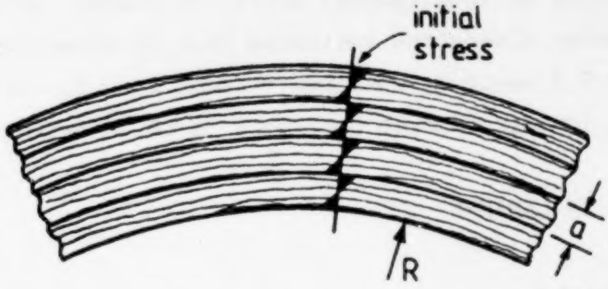
6. EFFECT OF CURVATURE ON BENDING STRENGTH

6.1 Effect on Longitudinal Stresses

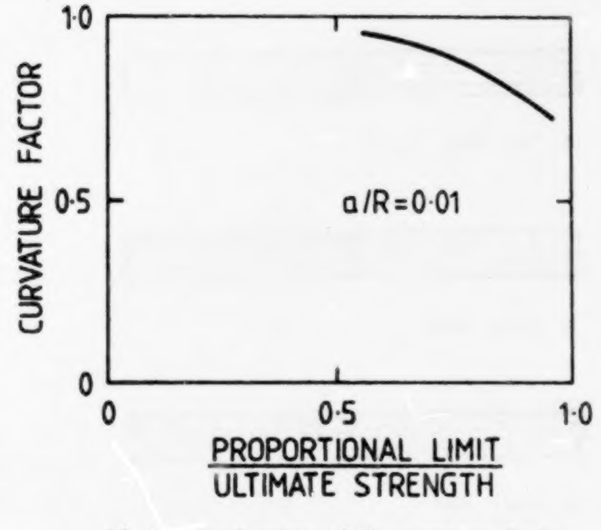
The effect of longitudinal stress on strength is described in terms of a curvature factor, denoted by CF and defined as follows

$$CF = \frac{\text{MOR of curved member}}{\text{MOR of straight member}} \qquad (6)$$

The effect of this curvature factor is illustrated in Figure 5 and is based on the reports by Finnorn and Rapari (1959), Hudson (1960), Kostukevich and Wangaard (1964), Wangaard _et al._ (1968.



(a) Initial stresses due to fabrication



(b) Measured values of the curvature factor

Figure 5 Effects of curvature on longitudinal strength

For a radius-to-thickness ratio of 100, the bending during fabrication can introduce an initial stress of up to 40 per cent of the ultimate strength of the laminae. The effect of this is to give a curvature factor as defined by equation (6) of about 0.8 immediately after fabrication; this relaxes to a value of 0.9 after a year or so.

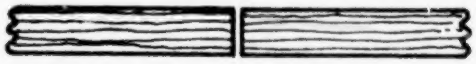
6.2 Effect on Transverse Stresses

As discussed elsewhere, the application of a bending moment to a curved member can introduce a transverse tension stress which can lead to failure by fracture of longitudinal splits or checks. An examination of some typical member dimensions indicates that an effective curvature factor of about 0.8 would be obtained through such failures if the members were to contain through cracks of 20 to 30 mm in length.

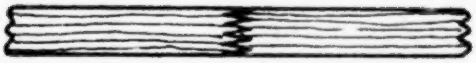
7. END-JOINTS IN LAMINAE

7.1 Types of Joints

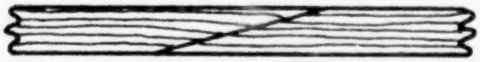
The three most commonly used end-joints in commercial practice are the butt, finger and scarf joints illustrated in Figure 6.



(a) Butt Joint



(b) Finger Joint



(c) Scarf Joint

Figure 6 Types of end joints

7.2 Butt Joints

The characteristics of butt joints have been discussed elsewhere. It has been found that in general it is uneconomical to place butt joints in tension members or withn the bottom third of beams; the reason is that to do so would necessitate increased member sizes due to the poor strength of butt joints.

7.3 Finger Joints

Currently this is the most successful type of end joint for commercial application. The machinery available for fabricating finger joints is quite effective and reliable. Typically the joints have a strength equal to that of structural timber having a grade ratio of about 0.4 and for design purposes may be considered to be equivalent to such timber.

If a finger joint fails to glue effectively, it acts as a butt joint with a consequent dramatic drop in strength. There would appear to be no quality control technique that will quarantee that the quantity of defective finger joints is a negligible proportion, such as less than one in 10,000. Consequently when finger jointed laminae are used for critical structural members it is recommended that every fabricated joint should be proof tested to ensure its structural integrity.

7.4 Scarf Joints

Correctly fabricated scarf joints have a strength close to that of clear wood and these joints used to be strongly favoured by the aircraft industry. However in the rough environments of factories that fabricate building products, it is difficult to fabricate satisfacory scarf joints having a scarf slope of less than one in 10; furthermore ineffective scarf joints act like butt joints, and the proportion of such joints tends to be larger than that found in the fabrication of finger joints. Consequently the use of scarf joints is not generally recommended for the fabrication of critical structural members.

7.5 Spacing of End Joints

The spacing of butt joints has a significant effect on the strength of glulam; this effect can be predicted accurately through the use of fracture mechanics theory and has been discussed elsewhere.

The spacing of scarf joints, if carefully fabricated, does not appear to affect the tension strength of glulam (Isyumov 1963). In theory the spacing of finger joints should also have a negligible effect. However in one experimental study Stickler and Pellerin (1971) found that the vertical stacking of finger joints lead to a loss in strength of 15 per cent; this was probably due to the fact that commercial finger jointing machines cannot produce sufficiently perfect joints so as to completely avoid the occurrence of stress discontinuities across joints.

8. FABRICATION OF GLULAM

8.1 Fabrication Methods

Glulam is fabricated with a great range of timber species. In general the difficulties of successful gluing increase with the density of the timber.

Cold setting glues are most commonly used, and three popular types are the following

- urea formaldehyde
- casein
- resorcinol

Urea glues are cheap but deteriorate in high temperature and high humidity environments, and some ureas have a questionable long term integrity. Casein glues are excellent in dry environments, but in humid environments are prone to attack by fungi. Resorcinol glues have the best all-round characteristics including stability in exposed environments; however they are also the most expensive of the three cold setting glues mentioned.

Hot setting glues such as the phenolics have an excellent stability in exposed environments. They are usuallly cured through the application of radio-frequency techniques.

Many additives to timber, such as preservatives and fire retardants, create difficulties in gluing and special fabrication techniques are necessary for effective gluing if these additives are impregnated prior to the laminating process.

8.2 Quality Control

Notionally, quality control should comprise the following 2 components:

- laboratory tests to assess the timber, the glue and the operating limits for successful fabrication
- on-line checks of the fabrication process to ensure that the acceptable range of the operating parameters are not exceeded.

Parameters of significance include the species, density, surface condition and moisture content of the timber; the mix temperature and pot life of the glue; and the temperature, pressure and time associated with the pressing operation. Many of these parameters are difficult to monitor continuously, and consequently it is usual to also specify quality control checks on samples of the finished product.

9. COSTS OF GLULAM

The following example of costs, assessed by one particular manufacturer, may be taken as indicative of the components of costs associated with the fabrication of glulam.

Item	Relative cost
Basic costs	
- timber	55
- UF glue	5
- labour	25
- capital	15
	100
Extras	
- end jointing	25
- resorcinol glue	15
- CCA treatment	15
- sealer and primer	5
- patching	20
- wrapping	5
	85

10. DURABILITY OF EXPOSED STRUCTURES

Glulam is frequently used for large exposed structures, and consequently their performance in exposed conditions is frequently a major concern. Experience has shown that even well glued members can delaminate alarmingly.

Probably the most useful information on this matter is to be found in the paper by Huggins and Aplin (1965) who undertook a survey of 57 Canadian bridges, fabricated from softwoods, and up to 30 years old. Aspects noted in the survey included the following:

(a) Creosote pressure treatment at a retention rate of 30 kg/m^3 proved to be completely satisfactory.

(b) Severe checking occurred if a moisture content differential of 5 per cent occurred between the surface and a depth of 60 mm.

(c) There was negligible checking on surfaces protected from wind and rain. A North-South orientation led to much worse deterioration than an East-West orientation.

(d) CCA treatment was associated with severe checking.

From the above, it would appear that the best protection against delamination and/or severe checking in exposed locations is obtained through impregnating the timber with an oil to deter the two-way movement of moisture through the surface of the glulam. Impregnation with oil based additives can be made either through conventional pressure treatment processes or through hot-and-cold bath methods. The latter process requires only the construction of a shallow, heatable bath and is particularly suitable for members of awkward shape that may not fit into commercial pressure treatment cylinders.

11. REFERENCES

Bohannon, B., and Moody, R.C. (1973). Evolution of glulam strength criteria. Forest Products Journal, Vol.23, No.6, June, pp.19-24.

Bohlen, J.C. (1972). LVL Laminated-veneer-lumber - Development and economics. Forest Products Journal, Vol.22, No.1, January, pp.18-31.

Bohlen, J.C. (1974). Tensile strength of Douglas-fir laminated-veneer lumber Forest Products Journal, Vol.24, No.1, January, pp.54-58.

Carruthers, J.F.S., and Burridge, M.S. (1964). The edge-tacking of laminated members. Wood, August.

Curry, W.T. (1967). Grade stresses for structural laminated timber. Special Report No. 15 (Second Edition). Forest Products Laboratory, Princes Risborough, Bucks., UK.

Dawe, P.S. (1965). Strength of finger joints. International Symposium on Joints in Timber Structures. London, UK, March.

Finnorn, W.J., and Rapari, A. (1959). Strength of curved laminated beams. Forest Products Journal, August, pp.248-250.

Freas, A.D., and Selbo, M. (1954). Fabrication and design of glued laminated wood structural members. Technical Bulletin No. 1069, USDA, Madison, Wisconsin, USA, February.

Hudson, W.M. (1960). The effect of initial bending on the strength of curved laminated timber beams. Wood (London), Vol.25, pp.234-236.

Huggins, M.W., and Aplin, E.N. (1965). Study of checking and delamination in glulam bridge members. The Engineering Journal (Canada), Vol.48, No.6, June, pp.44-51.

Huggins, M.W., Palmer, J.H.L., and Aplin, E.N. (1966). Evaluation of the effect of delamination. The Engineering Journal (Canada), February, pp.3-12.

Isyumov, N. (1963). Is minimum scarf joint spacing needed in glulam? Canadian Wood Products Industries, June.

Koch, P. (1973). Structural lumber laminated from 1/4-inch rotary-peeled Southern pine veneer. Forest Products Journal, Vol.23, No.7, July, pp.17-25.

Koch, P., and Woodson, G.E. (1968). Laminating butt-jointed, long-run Southern pine veneers into long beams of uniform high strength. Forest Products Journal, Vol.18, No.10, October, pp.45-51.

Kostukevich, C., and Wangaard, F.F. (1964). Curvature-stress factor in laminated wood beams. Forest Products Journal, January, pp.44-50.

Marx, C.M., and Moody, R.C. (1981). Bending strength of shallow glued-laminated beams of a uniform grade. Research paper FPL 380, April. Forest Products Laboratory, USDA, Madison, Wisconsin, USA.

Marx, C.M., and Moody, R.C. (1981). Strength and stiffness of small glued-laminated beams with different qualities of tension laminations. Research Paper FPL 381, May. Forest Products Laboratory, USDA, Madison, Wisconsin, USA.

Nemeth, L.J. (1967). Allowable working stresses for vertically laminated timber beams. Forest Products Journal, Vol.17, No.4, April, pp.23-30.

Pearson, R.G. (1974). Application of fracture mechanics to the study of the tensile strength of structural lumber. Holzforschung, Vol.28, No.1, February, pp.12-19.

Peterson, J., and Noziska, D. (1973). The tensile strength of laminated members. Forest Products Journal, Vol.23, No.11, November, pp.50-51.

Selbo, M.L. (1963). Effect of joint geometry on tensile strength of finger joints. Forest Products Journal, September, pp.390-400.

Standards Association of Australia (1975). Australian Standard 1720-1975. SAA Timber Engineering Code. Sydney, Australia.

Stickler, and Pellerin (1971). Tension proof loading of finger joint for laminated beams. Forest Products Journal, Vol.21, No.6, June, pp.19-24.

Sunley, J.G., and Dawe, R.S. (1965). Strength of finger joints in timber structures. The Timber Trades Journal, March.

Wangaard, F.F., Woodson, G.E., and Murray, M.R. (1968). Species response to prestress in curved laminated wood beams. Forest Products Journal, Vol.18, No.1, January, pp.49-56.

Westman, E.F., and Nemeth, L.J. (1968). Single ply vs. 2-ply laminated tension values. Forest Products Journal, Vol.18, No.8, August, pp.41-42.

Woodhead, D.W. (1972). Evaluation of a continuous laminating system. Australian Timber Journal, Vol.38, No.7, August, pp.26, 29, 31.

Woodson, G.E., and Wangaard, F.F. (1969). Effect of forming stresses on the strength of curved laminated beams of loblolly pine. Forest Products Journal, Vol.19, No.3, March, pp.47-58.

ADHESIVES FOR TIMBER

R. E. Palmer[1]

SUMMARY

Adhesive bonding (gluing) is but one of the processes used in the manufacture of certain structural components.

Bonds can be categorised according to service requirements as structural, temporary structural or non-structural. A structural bond has the strength and durability of the wood being bonded. Thus structural design incorporating bonded components can be based on wood strengths and the bond does not constitute a fault or weakness.

Bond testing is based on a comparison of bond strength with wood strength. Sample joints are subjected to a specified water soak treatment and then broken apart. Bond quality is expressed in terms of the area of broken fibre as a proportion of total bonded area. The load at break is regarded as only of secondary importance. The water soak treatment serves either to simply weaken the joint or in its more severe forms, to provide an accelerated ageing regime.

Bonds produced using a phenolic type adhesive in which test specimens give high wood failures after a 72 hour boil can be expected to last for 25 years in full weather exposure or for an indefinite period under protected conditions. No statement of this kind can be made for any other adhesive type.

There are no adequate non destructive methods for testing wood bonds. Stresses imposed during proof loading can only be a small fraction of those required to bring about bond rupture.

Since bonded structural components are essentially untested, then the specifying engineer must rely on the skill and integrity of the component manufacturer.

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In choosing a reliable manufacturer national standards can be of some assistance provided that they include a relevant set of test methods and minimum personnel and equipment requirements for manufacturers. There should also be some regulatory authority with the power and determination to deregister manufacturers who do not strictly adhere to the requirements of the standard. Few countries satisfy all these conditions. Test methods for hardwood bonding are generally inadequate.

Successful bonding of structural components requires good control of process variables such as time, temperature, pressure, wood quality and moisture content. This level of control cannot be attained on most building sites so that any on-site gluing must be of a non-structural nature.

INTRODUCTION

This lecture is an attempt to present what is essentially an Australian philosophy on structural wood gluing. This philosophy has been developed over many years because of the need to bond a large number of both hardwoods and softwoods covering a wide density range primarily in plywood manufacture but also in solid wood gluing for glulam production. Such material can be put into service in a wide range of climatic conditions ranging from Cairns to Alice Springs to Hobart. This is in contrast with the developed countries of the northern hemisphere where bonded structural components are manufactured from a rather smaller number of relatively low density species which are mostly softwoods of fairly similar gluing characteristics. The successful gluer in Australian industry tends to be somewhat of an experimenter who must be able to cope with a wide range of production variables only some of which are under his control. Clearly this does not apply to particleboard production to the same extent where the process is more in the nature of mass production. Also particleboard finds only a fringe structural application in its use as domestic flooring.

THE ADHESIVES

An adhesive is a substance capable of holding materials together by surface attachment. The materials held together by an adhesive are termed adherends. For many centuries the principal wood adhesive was animal glue so that the terms gluing and adhesive bonding of wood have become synonymous.

The wood adhesives of greatest commercial importance are those based on formaldehyde, i.e. urea-, melamine-, phenol-, tannin-, and resorcinol-formaldehyde (UF, MF, PF, TF, and RF). Also of importance are those based on polyvinyl acetate (PVA) which have largely replaced UF and animal glue in furniture and interior joinery applications. Elastomerics are used primarily for interior panel and trim attachment but are likely to become more important in adding to the stiffness of structural assemblies. Epoxies are expensive and can vary widely in performance because of the large number of possible variations in formulations. They tend to be used in special applications where cost is not a major consideration. A brief introduction to the use of these adhesives is given in the attached CSIRO information sheet. They are dealt with more fully in the United States Department of Agriculture publications listed under further reading.

WOOD AS AN ADHEREND

Certain characteristics of wood make it a unique adherend. These have been discussed in some detail in previous lectures and so are only briefly summarised below.

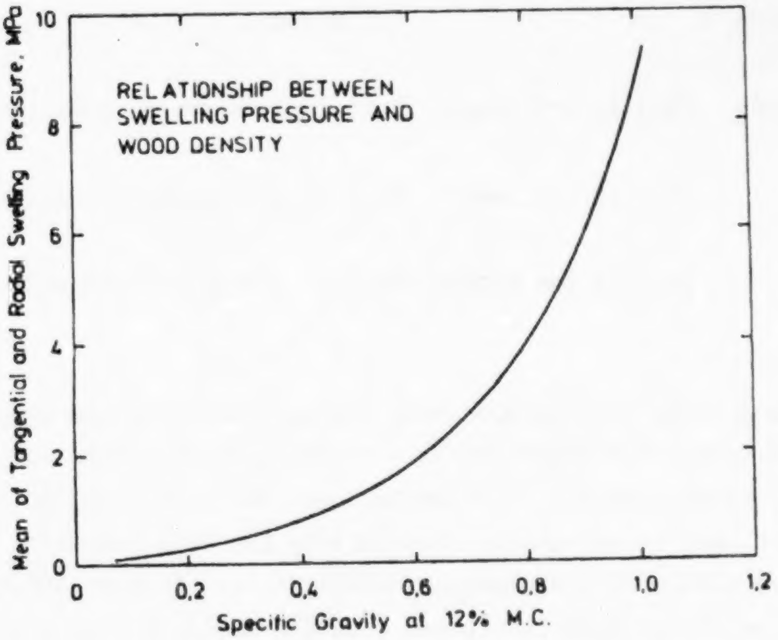
Wood has properties that are highly directional due to the orientation of its constituent fibres.

Wood is hygroscopic, i.e. its moisture content depends on the temperature and relative humidity of the surrounding air. Both its dimensions and its propensity to absorb water are dependent on its moisture content.

Wood is porous. The art of gluing lies in being able to control the flow of glue into the first few layers of cells just prior to cure. Because of its porosity wood offers little protection to the glueline from the surrounding environment. Variation in wood density arises out of variation in degree of porosity.

Wood density can vary by a factor of about 10. Strength properties generally increase with density. Swelling force, i.e. the force needed to prevent expansion of wood during a moisture content increase also increases with wood density. See Fig. 1. (after Kingston and Perkitny (1972)). The glueline constitutes a link in a mechanical system. The external forces capable of being applied to the glueline and the internal forces generated as a result of wood moisture content fluctuations both increase with wood density. Since porosity decreases with increase in density then the difficulty in producing bond strengths that match wood strengths increases with increase in density.

FIG. 1



DURABILITY CONSIDERATIONS

The use of adhesives in building construction depends on a thorough knowledge of adhesive long term durability. The exposure trials carried out by Knight (1968) over a period of thirty-five years have provided what still constitutes the most useful and extensive body of data available on wood adhesive durabilities. A summary is given in Table I.

TABLE I

Performance of Various Types of Wood Adhesive in terms of end use environment and actual or estimated life

Chemical type of adhesive	Exposure and performance		
	Full exterior	Semi-exterior and damp interior	Dry interior
Resorcinol and Phenol resorcinol formaldehyde	Expected life of 25 years	Indefinitely long	Indefinitely long
Melamine fortified urea-formladehyde	Fail in 5-10 years	Estimated life of 10-20 years	Indefinitely long
Urea formaldehyde	Fail in 2-5 years	Fail in 5-10 years	Indefintely long
Casein	Fail in 1-2 years	Fail in 2-5 years	Indefintely long
Animal	Fail in few months	Fail in a year	Indefintely long

In these trials a range of test specimens was used with plywood being the predominant type. The effective life of the bond was taken as the time to reach a certain degree of delamination. While this end point is obviously beyond what is acceptable structurally the data indicates that durability is relatable to the chemical nature of the adhesive and the environment to which the bond is exposed. On the basis of this and other accumulated data there is only one chemical type of adhesive that can be used in any structural application where it is possible to use wood. This is the phenolic type.

There are two other adhesive types that can be used in the unlikely event that it is possible to guarantee that the bond will not be subjected to conditions of high or widely flucturating relative humidity throughout the life of the structure. The first is casein which has a high reputation for structural integrity in interior conditions in temperate climates. It can be used in wooden aircraft but there are strict design requirements so that all joints are self-draining and such aircraft must be kept in a hangar when not in use. When saturated with water the strength of a casein bond can fall to as low as 20% of initial dry strength. This is largely recoverable provided that drying takes place within a short time. Prolonged high humidity conditions lead to breakdown by biological attack. While attempts have been made with varying success to reduce the susceptability of casein adhesives to biological breakdown by the addition of various preservatives the service restrictions remain. The other type of adhesive is that based on melamine. Straight MFs have a limited application because they have a very short shelf life and are less durable than straight PFs even though they are of similar cost and have similar high temperature curing requirements. Their use in Australia is largely confined to the production of Class I particleboard. A more common form is a melamine fortified urea-formaldehyde (MUF) adhesive which can have a long shelf life, is much cheaper than the equivalent phenolic type and can be cured under ambient conditions. Its breakdown mechanism is by hydrolysis probably accelerated by the fluctuating stresses associated with normal moisture content changes in service.

Since large variations in formulation are possible within any chemical type, how then does one pick a potentially durable formulation and know that it is being used in such a way as to produce a durable bond? In practice there are two types of test. The first is a simple strength test usually in shear or cleavage. The second type is a strength test after some accelerated aging regime that has been shown from long term durability trials to bring about a strength loss in a way related to that which would occur in service. The test requirements for plywood adhesives in a soon to be published Australian standard are given in Table II. The test for plywood is a cleavage test in which bond strength is compared with wood strength. On cleavage the break must occur predominantly in the wood regardless of wood density. The ability of other adhesives to pass these tests is no guarantee of their long term durability. The lack of a similar set of tests for other adhesives such as epoxies, polyurethanes, cross linked PVA's, acrylics, etc, rules out their use in structural applications at the present time.

TABLE II

AUSTRALIAN PLYWOOD BOND TYPE CLASSIFICATION

Bond Type	Service condition	Adhesive type (see Note 2)	Water soak treatment (prior to testing to Clause 7.2 of AS 2098.2)	
			Temperature	Time
A	Type A bonded plywood panels can be used in applications involving extreme long-term exposure to weather or wet or damp conditions and/or long-term structural performance requirements without glueline breakdown or glueline creep. Examples: Structural plywood flooring, highway signs and marine plywood.	Synthetic phenol-formaldehyde, natural polyphenol-formaldehyde or mixtures thereof	100°C	72^{+1}_{-0} h or 6 h at 200 hPa steam pressure
B	Type B bonded plywood panels can be used in applications involving short-term (not more than two years) exposure to weather or wet or damp conditions and/or short-term structural performance without glueline breakdown or glueline creep. They can also be used for applications involving long-term exposure to protected exterior non-structural environments. Examples: Concrete formwork, exterior door skins.	Melamine-urea-formaldehyde	100°C	6 h $^{+5}_{-0}$ min
C	Type C bonded interior plywood panels can be used in applications involving full protection from the weather or wet or damp conditions. They can be used in applications involving long-term exposure to generally high humidity and short-term exposure to extremely high humidity. Examples: Interior panelling in geographical locations which have prolonged periods of high humidity, panelling in a bathroom.	Urea-formaldehyde	70 ± 1°C	3 h $^{+5}_{-0}$ min
D	Type D bonded plywood panels can be used in interior applications fully protected from the weather or wet or damp conditions. They can be used for interior applications involving long-term exposure to medium humidity with occasional exposure to high humidity. Examples: Furniture, interior wall panelling.	Extended urea-formaldehyde	15°C to 20°C	16 h to 24 h

NOTES:
1. Experience has shown that Type A bonded plywood will withstand complete exposure to the weather for 20 years or maintain its integrity in structural situations for 50 years without glueline breakdown or glueline creep.
2. With adhesives of other chemical types, compliance with the test requirements alone would not indicate equal durability, and confirmation by actual service trials would be required.

PHENOLIC ADHESIVES

In view of the fact that only adhesives of the phenolic type can be used in unrestricted structural applications then the remainder of the lecture will be largely confined to these with some mention of the closely related melamine types. The traditional phenolic adhesives are the straight alkali catalysed PF's for use in hot press applications such as plywood or particleboard manufacture. Acid catalysed phenolics can be cured at room temperature but the residual acid continues to degrade the wood and can lead to early failure. Resorcinol is closely related chemically to phenol but reacts much more rapidly with formaldehyde. RF adhesives can thus be cured at ambient temperatures and so be used for the manufacture of glulam, finger jointed material and various fabricated components. Wattle tannin is a natural polyphenol and TFs can be used where straight PFs are used. Resorcinol is an expensive chemical and is often partly replaced by tannin or phenol. However, phenol resorcinol-formaldehyde adhesives (PRF) require a higher curing temperature than straight RFs. Of the formulations available in Australia the minimum glueline temperature for the cure of RFs is taken as 20°C whereas it is 40°C for a PRF. The formulation of adhesives is a science in itself and well outside the scope of most end users.

TESTING AND DURABILITY PREDICTION

Experience with phenolics over many years has shown that bonds prepared under the same conditions as test pieces that exhibit high wood failures both initially and after a 72 hour boil can be expected to remain structurally sound for at least 25 years even under more adverse conditions in which case the durability of the wood itself is more likely to be the limiting factor. These three requirements of chemical type, 72 hour boil and high wood failure provide the basis of the long term durability prediction. In the case of plywood the 72 hour boil test is used both as a quality control test and an accelerated ageing test. In glulam or finger joint manufacture (and in plywood in the United States) a vacuum-pressure soak treatment (VPS) is applied to the test specimen to provide a slightly more severe initial or quality control test. In a vacuum pressure soak treatment the specimen is immersed in water in a pressure vessel. A vacuum is drawn to withdraw the air from within the wood and then a pressure applied in order to bring about saturation. Times and pressures vary with different standards. This has the effect of placing some stress on the glueline and possibly eliminating hydrogen bonding effects. The 72 hour boil test is not a routine test but used only to establish that the particular adhesive formulation will have the required durability when used to bond the species to be used in actual manufacture. Some formulations are capable of producing bonds only with low density species. Also, the presence of preservatives and some wood extractives can interfere either with surface wetting characteristics or with the curing reaction of the adhesive. In the case of poor wetting characteristics, such as with teak, low wood failures are obtained when specimens are broken either in the dry state or after a VPS. Where the problem is one of interference with the curing reaction then low wood failures show up after a 72 hour boil. This is known to occur with some eucalypts because of the presence of hydrolysable tannins. It also occurs where CCA preservative treatments have been applied. In general CCA treated timbers cannot be glued consistently on a commercial basis.

In many glulam and finger-jointing standards the bond is assessed by determining the amount of delamination visible after a standardised VPS and drying regime. Australian experience has shown this to be quite inadequate. It can be demonstrated that where delamination occurs in such a test then there is zero wood failure if the bond is cleaved in the wet condition after a VPS. Hardwoods with a density of up to 900 kg/m^3 are glued commercially in Australia. Such beams that have delaminated in service are ones that have been accepted on the basis of high initial shear strength values in the absence of high wood failures.

A complicating factor in the testing of hardwood joints is the low permeability of some hardwoods to liquid water. In some Australian hardwoods the penetration of liquid water under a standard VPS treatment can be of the order of millimetres and so little stress is placed upon the glueline. The proposed test specimen in the Australian glulam standard presently under revision is a 25 mm slice of beam with a saw cut extending 10 mm into the glueline on the cutface. After a VPS treatment the gluelines are cleaved wet and the wood failure estimated after drying.

While the gluing considerations in plywood and glulam production are thought to be reasonably well understood this is not the case for finger jointing. Finger joints are generally tested in bending and test results are usually highly variable. There tends to be an inverse relationship between load to failure and amount of wood failure. Those specimens that break at a load consistent with the average bending strength for the species tend to exhibit quite low wood failure particularly in the higher density species. A suitable test specimen has not yet been adopted that allows for the low water permeability of some hardwoods. Where laminated beams are tested to destruction the break frequently begins in a finger joint where one is present in the lower laminate. Given that all the wood bonds lose strength to some extent with time and that those exhibiting low wood failure do so more rapidly then there must be some doubt about the durability of finger joints particularly in critical tension situations. These considerations have led some Australian glulam manufacturers to either use scarf joints in the outer laminates or to regard finger jointing purely as a materials handling aid and to design beams on the basis that all joints act as butt joints. The fact that there have been no known failures of beams in service due to finger joint failure is probably more a comment on a tendency to overdesign rather than on gluing practice. Clearly there are some species that can be finger jointed to a consistently high quality. At present the main outlet for finger jointed material in Australia is for wall studs and bottom plates. A very thorough knowledge of the quality of production should be obtained before considering the use of finger jointed material in situations any more critical than these.

While scarf jointing does not lend itself to automation in the same way as does finger jointing it is possible, with care, to consistently produce high strength, high wood failure joints.

SPECIFICATION

Durability considerations allow structural design incorporating bonded components to be based on wood strengths. These considerations also simplify test methods in that the wood has already been selected on the basis of strength. Proof testing provides virtually no information on bond quality and the only meaningful tests are destructive ones. This can be shown by considering a finger joint in timber graded as F17. A reasonable proof load for such a joint would be 2.5 times basic working stress or 42.5 MPa. The species average for clear material is likely to be in excess of 100 MPa. The modulus of rupture at the joint would need to be in the region of 80 MPa in order to exhibit significant wood failure. This is well in excess of any proof load.

Except in the case of plywood where a simple wetting treatment can be used to show up areas of delamination, bonded components are essentially untested. The specifying engineer must then rely on the skill and integrity of the component manufacturer. Many countries have industry organisations such as the Plywood Association of Australia who run a quality control scheme to ensure that any product bearing their stamp will conform to the standard to which it is produced. Such an organization must be prepared to withdraw the stamp of any manufacturer who consistently fails to meet the requirements of the standard. Where no such organisation exists then it is necessary for the engineer to satisfy himself that a given manufacturer is capable of consistently producing components to the rigorous standards required for structural applications. Clearly the engineer cannot be expected to be an expert in adhesives, wood technology and production engineering. This is the province of the manufacturer.

In assessing the capabilities of a manufacturer the first step is to determine whether he has the appropriate personnel and production and testing equipment for the purpose. The Canadian Standard 0177 Qualification Code for Manufacturers of Structural Glued-Laminated Timber provides a useful check-list in such an assessment. Production should then be sampled on a regular basis and tested to destruction according to the principles previously outlined.

Finally, successful structural bonding depends on close control of a range of variables such as time, temperature, pressure, wood quality and moisture content. The required level of control is unlikely to be attainable on a building site. Thus structural bonding will continue to be a manufacturing process rather than part of the construction process in the foreseeable future.

REFERENCES

Kingston, R.S.T. and Perkitny, T. (1972). On the relationship between the active swelling pressure of wood and passive compressibility by external forces. Holz als Roh und Werkstoff 30 (1), 18-28.

Knight, R.A.G. (1968). The efficiency of adhesives for wood. Ministry of Technology, For. Prod. Res. Bull. No. 38, HMSO London.

FURTHER READING

Gillespie, R.F., Countryman, D. and Blomquist, R.F. (1978). Adhesives in building construction. US Department of Agriculture. Agriculture Handbook No. 516.

Houwink, R. and Solomon, G. (1965). Adhesion and Adhesives. Elsevier. 2 Volumes.

Selbo, M.L. (1975). Adhesive Bonding of Wood. US Department of Agriculture Forest Service Tech. Bull. 1572.